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CONSTRUCTION ON PERMAFROST

(Stroyitelstvo v usloviyakh vechnoy merzloty)

by

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PREFACE

This work is devoted primarily to problems of construction of industrial and public structures under conditions of permafrost. Because of the unique environment, such engineering activity encounters many complications and often involves numerous misunderstandings and miscalculations. Knowledge and experience in this field are as yet inadequate. Moreover, a considerable part of the knowledge gained has not been available to the general construction profession and is therefore seldom utilized.

It is essential to propagate the available information about permafrost and to invest persistent and continuous effort to widen the scope of scientific and practical knowledge in this field. However, it is not judicious to bide time until all aspects have been investigated and tested. An immediate solution of numerous problems pertaining to construction in permafrost regions is imperative because such construction is currently in progress and cannot be postponed until various aspects are investigated, clarified, and verified. Construction is proceeding, and the structures are essential to effective development of the country.

It is the duty and responsibility of the engineers to assure normal erection and functioning of such structures even though current knowledge in this field is limited. Accordingly, in spite of the numerous factual gaps, it is necessary to summarize the available data as adequately as possible and thus help the engineers to overcome, as least partially, the difficulties encountered in permafrost regions.

The present work is the first one entirely devoted to industrial and public construction since the available studies of permafrost deal with either general frost science or railway construction. Therefore, it is of limited scope and probably contains inaccuracies and inadequate interpretations; these are justified by the lack of precedents and the complexity and obscurity of the problems involved. Its content is determined by its objectives, which are to acquaint engineers with permafrost as an entirely unique field of construction, to establish the peculiarities of construction on permafrost, to analyze the available theoretical and experimental data for application, and to indicate lines of further research that may assure success in a given case.

This book is presented in simplified form in order to make it valuable to both the specialists in this field and to persons of other professions and different degrees of training.

Leningrad, 1941 The Authors

ABSTRACT

This book contains general data on permafrost and characteristics of deformation of structures erected in permafrost regions. The surveys, design methods, and construction procedures involved are described in detail. The book deals primarily with industrial and public structures.

The book is intended for engineers and construction technicians working in permafrost regions but may be of value also to persons engaged in related professions.

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INTRODUCTION

Intensive efforts to exploit the vast, rich regions at the north and east of the Soviet Union (USSR) are accompanied by a boom in industrial, public, and road construction in unpopulated and relatively unexplored regions of extensive permafrost.

Permafrost often greatly complicates construction, involves considerable expenditure of money, time, and energy, and occasionally results in relatively irremediable errors that lower the tempo and effectiveness of the enterprise. A careful study of construction conditions in permafrost regions reveals that most complications and failures were and still are caused by the fact that the uniqueness of these regions is not taken into consideration because of inadequate knowledge of permafrost, its properties, and related phenomena. It is known that construction results were more successful and the structures did not deform whenever permafrost was taken into consideration, its phenomena studied under given conditions, and corresponding measures taken.

Proper construction on permafrost requires special knowledge of frost phenomena in general and thorough study of the actual conditions at the site involved. Nevertheless, in spite of numerous failures, deformations, and even collapses of completed structures, the engineers often still lack any clear understanding of the tremendous importance of knowing the nature of permafrost, of thorough analysis of local conditions, and of the need to coordinate design and construction methods with the specific local manifestations of permafrost and the corresponding circumstances.

Often engineers apply methods of design, construction, and survey of structure sites, which have been firmly established and were found absolutely justified under nonpermafrost conditions, to new conditions such as those of permafrost conditions, without taking into consideration or with inadequate consideration of the fact that the new conditions require a special approach. Of course, the results of such an attitude readily manifest themselves; deformations occur soon, and often it involves the need for major reconstruction or complete loss of the structure. It is true that knowledge of both permafrost and construction practice on permafrost is

inadequate. Nevertheless, since a certain amount of knowledge and experience is available, it is essential that these scientific and experimental data be judiciously applied in all cases.

Thus, the primary task of every engineer working under these interesting and complex conditions is to become better versed in permafrost, its characteristic phenomena, and the corresponding construction aspects. The average engineer often finds this task extremely difficult, since adequate knowledge of permafrost implies familiarity with a number of subjects: frost, hydrology, geology, soil science, meteorology, etc. The available data is scattered in numerous sources that cannot be readily utilized. Other sources were not intended for construction engineers; therefore, they contain unnecessary or relatively unimportant material, while certain major aspects are omitted. Finally, some works are obsolete or contain poor material.

Consequently, it is both interesting and valuable to endeavor to prepare a work that would enable construction engineers to familiarize themselves with permafrost and construction on it. A sufficiently complete and detailed, even though not profound, familiarization would serve the engineers as a firm basis for further development of their understanding in these fields and for an intelligent approach to the tasks of surveying, design, and construction under permafrost conditions.

In addition, considerable space is allotted to the characteristics of permafrost and its regions. Frost science specifies construction methods and means to prevent structural deformation; therefore, the chapter dealing with permafrost and its properties is extremely important and necessary for the subsequent chapters.

The deformation of structures under permafrost conditions should be given special attention, since examination of such deformation determines the effect of permafrost on structures, outlines the proper course for engineering effort, and clarifies as well as illustrates the phenomena resulting from occurrence of permafrost.

Measures against deformation of completed structures are complicated and often very expensive. On the other hand, deformation can frequently be avoided if the given construction conditions are thoroughly

considered. Stable construction on permafrost depends primarily on proper and adequate study and survey of the site involved. Advance and intelligent selection of the construction site, based on knowledge of permafrost, would prevent probable structural deformation. Since survey and analysis of the construction site yield extensive specific information, this aspect is fully developed in the present work. Proper selection of the site is the best guarantee of structural stability. It need be emphasized that effective construction and normal functioning of structures under permafrost conditions can usually be achieved only by applying the following measures: proper selection of the site after careful surveys and studies, proper selection of layout and design of the structures, use of suitable materials, proper execution of the work, and finally, appropriate operation of the completed structures.

The present work is of limited scope and discusses the problems primarily from the standpoint of construction; it deals mainly with industrial and public structures, and mentions other types of structures only in connection with the former or when necessary.

The last chapter of this book is devoted to aspects of experimental and analytical research. Although there are numerous permafrost experiment stations and considerable sums are used for experimental and scientific research, the organizations involved do not as yet have a unified plan of activity. In most cases, these stations are under the jurisdiction of different departments, and often engage in studies that are only indirectly related to construction.

In order to obtain timely solutions to important and as yet moot problems, it is necessary to unify the efforts of these stations and to conduct a series of simultaneous and analogous investigations. In this way valuable, mutually corrective, and supplementary data can be rapidly obtained.

Chapter VII discusses those problems which could be solved relatively readily but the solution of which requires experimentation under various conditions and in quantity sufficient to reduce the effect of chance and probable errors.

CONSTRUCTION ON PERMAFROST

CHAPTER I

GENERAL INFORMATION AND DATA ON PERMAFROST

A. Basic Definitions and General Concepts

Permafrost or permanently frozen ground refers to a layer of ground located some distance below the natural ground level and having a negative temperature lasting continuously over an indefinite time interval varying from two years to tens of thousands of years [1, page 30].* Thus, the term permafrost may be applied to any ground having a negative temperature for more than two years, regardless of its type, composition, moisture content, or phase (that is, the state of the water in this ground), and regardless of whether or not it contains water, whether it is friable or well cemented by frozen water, whether it is rock, gravel, sand, loam, slud**, etc.

The term permafrost is conditional and by no means implies a permanently or eternally frozen state of the ground. The term specifies only that the principal body of the frozen ground has existed since prehistoric times, which is proved by the fact that remains of prehistoric animals are found in permafrost.

In reality permafrost, which usually occurs in definite regions, does not remain unchanged but undergoes considerable change within relatively short time intervals. A rise or fall in permafrost level may occur, due to a number of accidental factors, such as a change in surface cover of the ground, a change in regime of ground water, or even shading of a given area. The instability of the permafrost level is a universally known fact which every construction engineer must take into consideration.

The permafrost layer is usually overlain by a ground layer in which the temperature changes periodically with the seasons of the year.

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* Numbers in brackets refer to the bibliography at the end of this report - M. P.

** Ground that behaves as a more or less viscous fluid - M. P.

As winter sets in, this upper layer of ground gradually freezes to a depth varying greatly with local conditions. In most permafrost regions, this seasonally frozen ground merges with the permafrost layer during December or January, forming a single frozen mass. In places where the permafrost layer occurs deep beneath the surface of the ground, a layer of unfrozen ground* remains between the seasonally frozen layer and the permafrost. The thickness of this talik ranges from a few centimeters to several meters.

In the spring, as the warm season of the year sets in, the seasonally frozen layer begins to thaw at the surface; the thawing progresses in depth with influx of warmth, until the warmth ceases and is replaced by a new cold wave. Deepest thawing usually occurs during September or October. The ground layer overlying the permafrost and alternately freezing and thawing each year is called "active layer," since the physical and dynamic processes affecting the strength and stability of structures occur primarily within this layer. Beneath the permafrost is a ground layer of positive temperature which increases with depth in accordance with the geothermal gradient effected by the heat flowing from the interior regions of the earth's core.

Thus, the permafrost is bounded at the bottom by a surface of constant temperature of zero degrees** and at the top by a surface having a temperature of zero degrees during maximal thawing of the active layer. These surfaces of zero temperature denote the upper and lower limits of permafrost. The distance between these surfaces constitutes the thickness of the permafrost layer. This thickness varies in accordance with the region; it ranges from a few centimeters at the southern boundary of permafrost to several hundred meters in the extreme north.

Ground with negative temperature but containing little or no ice, so that it is not adfrozen and stays loose, is termed "dry permafrost" to distinguish it from ordinary permafrost. Such ground usually

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*Talik.- M. P.

**Centigrade temperatures and metric system units are used throughout this text. - M. P.

consists of rocks, gravel, or sand.

The term "degradation of permafrost" denotes a process whereby the permafrost disappears due to natural or artificial causes. Permafrost in a given region may be regarded as undergoing degradation if the temperature of the ground gradually rises from year to year, which can be determined from careful observations of the temperature regime of the ground. Unfortunately, the degree and rate of degradation are unknown.

The origin of permafrost is, as yet, not sufficiently clear. Most students of this problem are inclined to believe that permafrost developed during a recent geological period. To the construction engineer, however, the question of origin of permafrost has no practical significance.

With regard to the moisture content of the permafrost and active layers, it is usually assumed that the ground composing these layers is supersaturated. The term "supersaturated ground" refers to ground containing more moisture than its water-holding capacity. Complete saturation is that state in which all the pores or spaces between soil particles are filled with water.

B. Geography, Morphology, and General Characteristics of Permafrost

According to M. I. Sumgin's latest data [1], the area of permafrost in the USSR is about 10^6 sq km, or about 47 per cent of the entire territory of the country. The distribution of the permafrost is shown on the map of Fig. 1. The northern part of the permafrost zone is occupied by a geographically continuous body of permafrost of considerable thickness. Farther south, there occur individual areas without permafrost, termed "islands of talik"; still farther south, near the southern boundary, there occur only islands and patches of permafrost within a continuous mass of thawed ground.*

The general character of permafrost is similar in all regions, except that the more southerly part of the permafrost body has a much

*Sporadic permafrost – M. P.

higher temperature, so that it differs somewhat from the northern permafrost. The approximate southern boundary of permafrost begins in the European USSR, north of Mezen,* and proceeds southward, crossing the Pechora River at the Arctic Circle; then, passing the Ural Range at latitude 65° N, it crosses the Ob River 250 km north of its confluence with the Irtish and slightly south of Berezovo; from there it proceeds toward Turukhansk and, crossing to the right bank of the Yenisei River, sharply turns southward, passing between Kansk and Krasnoyarsk; then, west of Irkutsk, it proceeds beyond the Soviet border; it reappears there in the USSR in the east, west of Khabarovsk, proceeds towards the mouth of the Selemdzha River, follows the left bank of the Amur River and, crossing this river near Lake Kizi, it ends at the Tartary Strait.

Two basic types of permafrost occur in a vertical cross section: (1) uninterrupted or continuous, consisting of a single layer of given thickness, and (2) layered, consisting of permafrost layers alternating with unfrozen layers. An example of the second type was observed at Igarka, where Eng. N. I. Bikov found five taliks in the permafrost within a borehole 62 m deep. Each of these basic types of permafrost may comprise two varieties. The first variety occurs when the ground layer above the permafrost completely freezes during the winter, so that the entire thickness, extending from the surface to the lower limit of the permafrost, forms a single frozen mass. This variety is designated as merging permafrost. The second variety occurs when talik always exists between the active layer and the upper limit of the permafrost. This variety is designated as nonmerging permafrost.

Accordingly, with respect to a vertical cross-section, permafrost may be classified as follows: continuous and merging, continuous and nonmerging, layered and merging, and layered and nonmerging. In accordance with this classification, the occurrence of permafrost can be represented by four schematic geological sections, as shown in Fig. 2.

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*The geographical names are in accordance with the 1947 World Atlas of the Encyclopedia Britannica. - M. P.

Due to several factors, the thickness of the permafrost layer varies greatly not only with the regions involved, but also within a given region. The thickness of the permafrost layer does not remain constant, but increases or decreases due primarily to changes in either the regime of the ground water or the state of surface cover and vegetation. It is obvious that the upper and lower limits of the permafrost cannot be planes, but have a random and irregular contour. The contour of the upper limit depends on the following aspects of the locality involved: topography, geographical location, nature of surface cover (forest, bushes, grass, moss, peat, plowed field, road, etc.), geological structure, and hydrological conditions. In other words, the contour of the upper limit has its own microrelief, differing from that of the ground surface. For this reason, the entire active layer does not freeze uniformly and simultaneously which, in turn, results in nonuniform swelling of the ground and formation of ground icings. Because of lack of adequate investigations, it is impossible at present to furnish any definite information regarding the contour of the lower limit of permafrost; in most cases, however, this aspect is of no significance as far as strength and stability of structures are concerned.

Even in the case of natural local regime, the upper limit of the permafrost is not constant and varies vertically because of numerous accidental factors. Human activity nearly always affects the position of this limit; it may lower this limit or raise it, depending upon circumstances, thus changing the thickness of the permafrost.

The following figures demonstrate the variation in the thickness of permafrost: 274 m (the lower limit has not been reached) at Amderma, in the extreme north of Siberia; more than 136 m (the lower limit has not been reached) at Yakutsk; 67 m at Bushuleya; 49 m at Petrovsk-Zabaikalsk; 50 m at Skovorodino; 71 m at Taldan; 1 to 2 m in the Taishet-Padun region; 46 m at Zilovo; 51 m at Mogzon. Thus, it is evident that the thickness of the permafrost greatly diminishes in the southern regions.

The thickness of the permafrost is of definite importance to construction engineers, since this thickness must be taken into consideration when structures are erected in accordance with the principle of permafrost conservation. If the permafrost layer is thin, it is improbable that it will remain permafrozen after the construction.

The temperature of the permafrost layer has not been sufficiently investigated as yet. Meanwhile, data about permafrost temperature are of considerable importance to those working at construction on permafrost, since determination of the cold reserve in the ground, which is related to the thickness of the permafrost, constitutes one of the major considerations for determining whether it is feasible to conserve the permafrost beneath the structure or to eliminate the permafrost by natural or artificial means. Moreover, temperature data are needed to determine the strength of the permafrost. Finally, such data are valuable for clarifying the aspect of settling. The lower the temperature of the permafrost, the stronger the ground and the easier to achieve permafrost conservation at the base of the structure.

It should be noted that a permafrost temperature of about -0.5° C or higher does not assure permafrost conservation at the base of a structure, while temperatures ranging from 0.0° C to -0.3° C may cause considerable settling of the frozen ground under load. The temperature of the upper portion of permafrost is not constant, but varies with the season of the year and several accidental factors.

Available data yield the following approximate geographical distribution of the permafrost in accordance with temperature [2, page 308].

No.	Region	Temperature in Degrees C
1	north of Mezen	≥ -0.5
2	lower reaches of the Usa River (right tributary of the Pechora River) to the Vorkuta Estuary	-1
3	from the northern Ural Range eastward to the Taz Bay	-3 to -5

No.	Region	Temperature in Degrees C
4	from Taz Bay to the Anabara River	-5 to -7
5	from the Anabara River to the Kolyma River at the 64th parallel	-6 to -8
6	Anadyr Estuary	-5
7	west of Lake Baikal, wherever permafrost occurs	≥-0.5
8	north of Lake Baikal to the Pacific Ocean, and south to the Soviet border	≥-1.5

These regions can be divided into three groups: northern, with permafrost temperature below -5° C; middle, with temperatures ranging from -5° C to -1.5° C; southern, with temperatures above -1.5° C.

Permafrost may comprise every type of ground. However, construction is most hazardous and difficult in the case of wet silt and loam which turn into slud when thawed; clay loam and fine sand fall into the same category. Coarse sand and, particularly, gravel or conglomerate drain readily and are more suitable, provided that they contain only small quantities of pure ice (ground ice). The quality of permafrost refers to its capacity for considerable settling during thawing beneath a structure and to the difficulty it involves during construction operations. Sandy loam and loamy sand usually occur in permafrost.

Permafrost is quite strong until thawed. The water saturating the ground and adfreezing to it cements it so strongly and forms such a solid mass that it is extremely difficult to excavate this ground. It is nearly impossible to use a pick or crowbar in permafrost; it becomes necessary to use explosives or to thaw the ground with bonfires or natural solar heat.

Silty ground of high moisture content thaws during the summer in the course of construction work and turns into slud; this interferes

with operations because traffic becomes impossible. Men, horses, wagons, and machines tend to sink into the liquid ground.

Data on moisture content of permafrost are of great importance. The relative moisture content is a factor in determining the operational procedures and the probable amount of settling under load and affects the strength of the frozen ground. With respect to mechanical properties of permafrost, the most favorable moisture content, expressed as per cent of voids filled with water, is approximately as follows:

for pure sand . . . . 75 per cent
for pure clay . . . . 45 per cent

With regard to the effect of moisture upon settling of thawed ground, even 30 to 35 per cent moisture, which corresponds to the water-holding capacity of common permafrost, constitutes a hazard. Settling is slight only if the moisture content is approximately two-thirds of the water-holding capacity. When referring to the moisture content of frozen ground, the term "moisture content" is usually replaced by the term "ice content" or "saturation with ice;" these terms have the same meaning in the case of permafrost, provided that they refer to moisture by weight and not by volume. The distribution of moisture in a permafrost layer usually is variable. However, the uppermost portion of the layer has a relatively higher moisture content. Accordingly, in the process of construction it is often sensible to penetrate the upper, highly wet layer of permafrost and to extend the foundations 1 to 2 m below the upper limit of the permafrost, basing the structure on ground that is less wet, even though frozen. Figure 3 shows a typical distribution of moisture in the permafrost and active layers.

The mechanical strength of frozen ground is of major interest to construction engineers. N. A. Tsytovich [2, page 150] has conducted careful and adequately detailed tests of this strength. These tests, as well as the tests by I. S. Vologdina and others, demonstrate that the compressive strength of frozen ground is generally much higher than that of common thawed ground, and depends upon temperature, moisture content or saturation with ice, and mechanical composition of the ground. ~~The results of most tests lead to the following conclusions:~~

1. The compressive strength of sandy ground is higher than that of clayey ground, all other conditions being equal.

2. The strength of the ground increases with increase in moisture content up to the water-holding capacity.

3. Any further increase in moisture content produces no increase in strength and often decreases the strength of the frozen ground.

4. The compressive strength of frozen ground increases with the lowering of temperature.

Table I, compiled from the data by N. A. Tsytovich, presents comparative values of mechanical strength of three types of frozen ground at various temperatures and relatively low saturation with ice. These values confirm the preceding statement regarding the effects of mechanical composition and temperature on strength. Since these are experimental data, they can be used only for purposes of approximate comparison and are unsuitable for accurate and final practical design.

Table I

COMPRESSIVE STRENGTH OF FROZEN GROUND

Classification by Texture	Temperature in Degrees C	Per Cent Moisture by Weight	Compressive Strength in Kg per Sq Cm
Sand	- 1	17	62
	- 3	17	78
	- 6	17	99
	- 9	17	118
	- 12	17	134
	- 20	17	152
Clay loam	- 0.3	43	6
	- 1.5	48	16
	- 5	21	44
	- 12	17	50
	- 0.3	59	5
Silty clay loam	- 1.1	20	28
	- 5	30	30
	- 10	61	35

Table II, likewise compiled from experimental data by N. A. Tsytovich, presents values of compressive strength of various types of ground at temperatures corresponding to those occurring in natural

permafrost (since temperatures lower than -4° C rarely occur in actuality) and at moisture contents approximately corresponding to full saturation with ice. Nevertheless, as in the case of Table I, even these data must be regarded as approximate.

Table II

COMPRESSIVE STRENGTH OF ICE-SATURATED FROZEN GROUND

Classification by Texture	Compressive Strength in Kg per Sq Cm		
	At Temperatures Not Lower Than -0.5 C	At Temperatures From -0.5 To -1.5 C	At Temperatures From -1.5 To -2.0 C
Sand	22	27	36
Loamy sand	11	22	—
Sandy loam	10	20	26
Clay loam	6	17	25
Silty clay loam	5	15	23

The mechanical properties of thawed ground differ radically from those of the same ground when frozen, particularly in the case of supersaturated loams and silts. Upon thawing, the strength of the ground frequently decreases to zero and extensive settling occurs.

The composition of the active layer, or of its lower portion at any rate, is similar to that of the permafrost. In the case of merging permafrost, the thickness of the active layer corresponds to the depth of maximum thawing reached at the end of the warm season. In the case of nonmerging permafrost, the thickness of the active layer corresponds to the depth of maximum freezing reached at the beginning of the warm season. The thickness of the active layer (layer of winter freezing and summer thawing) depends on several factors; the major factors are: climate, composition of the ground, vegetation, and moisture content. The depth of the active layer is greater in the south than in the north, all other conditions being equal. It is greatest in the case of sandy and gravelly grounds, is medium in the case of loamy grounds, and is smallest in the case of peat bogs. Thick moss cover greatly reduces the depth of the active layer; in the extreme north, for example, this cover limits the depth of the active layer to about 20 cm. The active layer is thicker in dry ground and thinner in wet ground.

Available observations yield the following approximate, avera values of thickness of the active layer:

Type of Ground	Geographical Location	Thickness in Mete
Sand	south of 55th parallel	3.0 to 4.0
	at the latitude of Yakutsk (62° N)	2.0 to 2.4
	at the Arctic Coast of Siberia	1.2 to 1.6
Peat bogs	south of 55th parallel	0.8 to 1.0
	in the extreme north	0.2 to 0.4
Loam		intermediate

It should be noted that these values were obtained from observations under natural, undisturbed conditions of the local regime; disturbance of this, which is inevitable whenever man appears and engages in econom activity, results in a change in the thickness of the active layer. Other conditions being equal, the active layer under bare surface is everywhere 0.5 to 1.0 m thicker than under natural surface cover.

Moisture distribution in the active layer is not uniform and depends on several factors. However, the lower portion of this layer, directly overlying the permafrost, usually has a relatively higher moisture content. The moisture content of the active layer is of great importance to the construction engineer, since it is the major factor which determines the adfreezing strength between the active layer and the foundations laid in this layer. In addition, swelling of the groun depends to some extent upon the moisture content of the active layer.

The temperature of the active layer depends entirely upon the time of the year and the thermal regime of the locality involved. The effects of mechanical composition and moisture content of the ground on temperature and particularly on the rate of heat transfer into the acti layer have been insufficiently investigated as yet. It is known, howev that wet ground freezes slower and to a shallower depth than dry ground because part of the cold entering the ground is consumed in the process of transforming the water in the wet ground into ice.

Of particular interest to construction engineers is the fact that both the minimum and maximum temperatures of the active layer lag

two to three months behind the corresponding mean monthly temperatures of the air. This fact is well illustrated by the data of Table III showing the mean monthly temperatures of the active layer at a permafrost station. At a depth of 2 m in the active layer, negative temperature occurs in December, while frosts set in at the end of September or at the beginning of October; on the other hand, positive temperature replaces the negative temperature in August, although frosts cease in May.

Table III

MEAN MONTHLY TEMPERATURES OF THE ACTIVE LAYER, IN DEGREES C

Depth in Meters	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
0.40	-12.9	-13.0	-9.1	-3.3	0.3	6.2	10.7	11.8	7.4	1.5	-2.2	-8.6
0.80	- 8.3	- 9.6	-7.6	-3.5	-0.9	1.6	6.4	8.5	6.4	1.6	0.0	-3.4
1.60	- 0.9	- 3.5	-4.3	-2.9	-1.4	-0.9	- 0.4	-1.3	2.4	1.0	0.1	0.0
2.00	- 0.4	- 1.9	-3.2	-2.6	-1.5	-1.0	- 0.7	0.0	1.0	0.6	0.1	0.0

In December or January the upper ground freezes only to a depth of 1.0 to 1.5 m, so that an unfrozen layer 0.5 to 1.0 m thick occurs between this active layer and the permafrost. This fact is of considerable importance to the construction engineer, since the unfrozen water in the talik tends to feed the ground that swells during freezing, while the water compressed by the frozen crust of the ground is thus under pressure. Moreover, this talik facilitates icing processes.

The active layer firmly adfreezes to the supports of structures during winter freezing. During deformation of the active layer, such as swelling, for example, the ground increases in volume and deforms the support by heaving it. When the adfreezing between the ground and the support is weak, the swelling layer separates from the support and exerts no effect on the structure. Accordingly, adfreezing strength is an important factor in construction because it affects structural stability.

The adfreezing strength between the active layer and various types of structural supports has not been adequately investigated. Available laboratory data convey only an indefinite idea of its magnitude, while actual field data are few and not fully reliable because

of inadequate verification. Nevertheless, the following may be regarded as established:

(1) Laboratory data greatly exaggerate the adfreezing strength, as compared with actuality.

(2) Adfreezing strength is not uniform throughout the depth of the active layer.

(3) Hazardous adfreezing occurs only within part of the active layer.

(4) Adfreezing strength increases with decreasing temperature of the ground, other conditions being equal.

(5) Adfreezing strength increases as moisture content increases to the point of complete saturation of the ground and is less than maximum when the ground is supersaturated.

(6) Adfreezing strengths of sand and sandy ground are greater than those of loam and loamy ground under conditions of similar moisture content.

Table IV, compiled from laboratory data by N. A. Tsytovich, shows the effects of temperature and mechanical composition on the adfreezing strength between ground and wood. The values shown are not necessarily indicative of actual conditions, but give a fair idea of the relative magnitude of adfreezing strength under various conditions. The data of this table are not suitable for design purposes but serve merely to illustrate the effects of composition and temperature on adfreezing strength. Examination of these data reveals that the last two magnitudes of adfreezing strength, referring to gravel in which the voids are filled respectively with ice and clay loam, are particularly unreliable for several reasons.

Firstly, Table IV shows that the adfreezing strength of pure gravel having a saturation coefficient of 79 per cent is only 0.9 kg per sq cm at a temperature of -9° C, while the adfreezing strength of gravel fully saturated with ice is 27.3 kg per sq cm, or 30 times as large at the same temperature. This is impossible, as the moisture content in the first case does not differ greatly from that of the other case. Secondly, it seems improbable that the adfreezing strength of pure gravel, which is determined primarily by the adfreezing strength between ice and wood (13.7 kg per sq cm), exceeds the latter strength to such an extent (27.3 - 13.7 = 13.6 kg per sq cm, or twice as large).

TABLE IV

TANGENTIAL ADFREEZING STRENGTH BETWEEN WOOD AND FROZEN GROUND OF DIFFERENT TEXTURE AT VARIOUS TEMPERATURES

Classification by Texture	Temperature in Degrees C	Per cent Moisture by Weight	Tangential Adfreezing Strength in Kg per Sq Cm
Ice	- 1	-	5.0
	- 5	-	6.2
	- 7	-	11.6
	-10	-	11.7
	-20	-	22
Clay loam	- 0.2	27	2.9
	- 1.5	27	2.9
	- 5.8	28	11.1
	-10.8	28	18.6
Loamy sand	- 0.2	12	1.3
	- 1.2	13	7.0
	- 5.2	15	19.6
	-10.7	14	24.7
Silty clay loam	- 0.2	30	3.6
	- 0.5	33	6.1
	- 5.7	34	10.6
	-10.3	33	14.3
Fine gravel (saturation coefficient 77 per cent)	-10	-	2.6
Medium gravel (saturation coefficient 79 per cent)	-10	-	0.9
Gravel saturated with ice	- 9.5	28	27.3
Gravel saturated with clay loam	-10.2	24	30.6

Similarly, it is equally improbable that the adfreezing strength of gravel in which the voids are filled with clay loam increases so much that it is almost twice as large as that of clay loam at the same temperature and moisture content.

It is obvious that the excessiveness of the magnitudes involved must be due to some chance factors. Nevertheless, they are of interest because they demonstrate that the adfreezing strength can be greatly increased when clay loam fills the gravel voids.

According to Tsytovich's recent data, the rate of load application has a considerable effect on the adfreezing strength. Test results show that adfreezing strength is two to three times greater at rapid loading than at slow loading. Tsytovich attributes this phenomenon to the plasticity of frozen ground.

C. Main Geophysical Properties of Permafrost Regions and Their Effect on Construction

The active layer is often quite wet and even supersatured. High moisture content of the upper layers of ground is typical for permafrost regions. When the permafrost layer occurs at considerable depth, the high moisture content of the active layer is due to the fact that the ground comprising this layer does not readily yield water to the adjacent layers. In other cases, supersaturation of the active layer is due to the facts that the upper layers of ground are underlain by rock at shallow depth and that this rock is either absolutely impervious or nearly so. Finally, whenever the active layer is directly underlain by permafrost or the permafrost occurs at shallow depth, so that thin talik forms between the active and permafrost layers, the upper layer of ground is supersaturated because the permafrost is practically impervious.

It is obvious that in the preceding cases and particularly in the last one, that is, wherever considerable summer precipitation occurs in permafrost regions, a large quantity of ground water is present, the water table is close to the ground surface, and the upper layer of ground is generally highly saturated. Under conditions of appropriate mechanical composition, this upper layer tends to turn into slud (saturated very fine sand or silt occasionally containing a considerable admixture of clay loam).

In addition to the gravity water that tends to percolate gravitationally into the ground to the upper limit of the permafrost, the low temperature of the permafrost causes continuous condensation of water vapor from the active layer on the permafrost surface when the active layer is in a state of thaw. Therefore, the portion of the active layer directly overlying the upper limit of permafrost

is particularly supersaturated.

Several physical and dynamic processes constituting the ma[illegible] causes of structural deformation occur in the highly wet active lay[illegible] during its freezing and thawing. When the water in the ground free[illegible] there occurs swelling which uplifts the overlying ground layers and, simultaneously, the foundations, columns, piles, and other structur[illegible] elements. Swelling occasionally attains a height measuring tens of centimeters.

Swelling may affect large areas. V. V. Elenevsky, an [illegible] who made a leveling survey of stumps in a marshland, has found that swelling caused some stumps to be displaced vertically a distance o[illegible] [illegible] cm. In addition to its direct effect on structures erected unde[illegible] conditions of considerable swelling of ground, the phenomenon of swelling is important in construction because the ground elevations established during topographic surveys in the winter will differ co[illegible] siderably from those established in the summer. Therefore, winter surveys nearly always result in large errors. Previous experience [illegible] that a difference in depth of frozen ground at opposite sides of a[illegible] foundation produces a resultant horizontal pressure which may attai[illegible] a magnitude sufficient to cause failure of a foundation not designe[illegible] to withstand such stresses.

As winter freezing sets in, the ground water becomes comp[illegible] between the impervious permafrost underneath the steadily growing [illegible] layer above. The resultant hydrostatic pressure is extremely high [illegible] times. Under this pressure, the water seeks an outlet to the surf[illegible] flows towards unfrozen areas, such as the areas beneath structures wherever the frozen layer is not yet thick, bursts through the upp[illegible] ground layer, pours out over the ground surface or under a structu[illegible] and forms ground icings.

The water in contact with the permafrost transfers its the[illegible] mal energy to the permafrost and tends to thaw it, while the perm[illegible] frost, absorbing the thermal energy of the water, tends to transfor[illegible] the water from a liquid into a solid, that is, into ice. As a re[illegible] sult, according to [illegible] Sumgin, active equilibrium occurs in each

permafrost region under given conditions between the volume of surface and ground waters in the liquid phase and the regime of the permafrost, that is, its geographical and geological continuity, its thickness and temperature. Therefore, it is essential that the construction engineer make a careful survey of the hydrology and hydrogeology of the given region and take the results into consideration when planning or constructing any type of structure.

According to N. I. Tolstikhin, the presence of permafrost specif the classification of ground water in the permafrost region into three mutually related types: suprapermafrost, subpermafrost, and intrapermafrost. Suprapermafrost water is ground water above the impervious permafrost layer. This water usually occurs either in liquid or solid state, depending on the season of the year; in cases when winter freezing (that is, the active layer) does not reach the upper limit of permafrost, this water remains liquid throughout the entire year. When the active layer freezes, the suprapermafrost water causes deformation of the ground surface, in the form of frost mounds, polygonal markings, etc. Intrapermafrost water occurs within the permafrost. The solid and liquid phases of this water remain stable with respect to time. In its liquid state, this water does not exist independently but links the suprapermafrost and subpermafrost waters. Subpermafrost water occurs beneath the permafrost, often at considerable depth. It is always liquid and under constant pressure. Consequently, under favorable conditions this water frequently bursts through the permafrost layer and emerges at the ground surface in the form of continually active springs. These springs generally do not freeze in winter and so cause formation of spring icings. Figure 4 shows one of the fountains which appeared near the weather station at Urusha in the spring of 1912; it adequately illustrates the presence of water under pressure and of moisture in the ground.

The ground water within the permafrost layer occurs also in solid form, that is, in the form of ice. This fact distinguishes this ground water from that occurring in regions where permafrost does not exist. According to Sumgin, the following are the types of ice usually occurring in the layer beneath the active layer:

(1) Imbedded ice, that is, ice that formed on the ground surface and subsequently became imbedded in the ground, which comprises:

(a) Glacial ice buried beneath sediments. This type of ice covers extensive areas and occurs on the Novosibirsk Archipelago, along the coast of the Arctic Ocean east of the Lena Estuary, and in the Lena-Aldan Watershed.

(b) Ice formed from accumulations of snow (snow banks) covered by later sediments. The areas of this ice are small and occur in the north.

(c) Ice of icings covered by ground. This type of ice occurs throughout the permafrost region. Stratification is its characteristic aspect. It covers relatively small areas, but its thickness is considerable.

(d) Lake ice covered with ground.

(e) River ice cast ashore and buried beneath sediment.

(f) Sea ice cast ashore and covered with ground.

(2) Ground ice, that is, masses of ice formed by water freezing in the ground proper, which comprises:

(a) Ice formed within icing mounds. Its areas are small.

(b) Ice formed by freezing of suprapermafrost water. Such ice occurs in relatively thin layers but covers extensive areas.

(c) Ice formed by freezing of intrapermafrost water.

(d) Ice crystals and thin veins formed by freezing of water in taliks that became permafrozen.

When imbedded ice or ground masses sat ated with ice are located at shallow depth and happen to thaw, the ground surface sags, the depression fills with surficial water, and a thermokarst lake is formed. These lakes are small and their size often varies due to continuing melting of the ice lenses and veins in the ground. Trees and bushes that sagged with the ground often are found in the water of thermokarst lakes. The presence of these lakes indicates that the given locality is unsuitable for construction because of its permafrost geology and that it is necessary to conduct careful surveys to determine the extent and location of the imbedded ice.

The task of conserving the imbedded ice in regions that are being developed by man is extremely difficult and uncertain, since the existence of such ice is largely determined by the established regime of the ground water and partly by the nature of the locality and its surface cover. It often melts in regions entirely undisturbed by man; the phenomenon is due to local factors not related to human activity. This is proved by the occurrence of the thermokarst lakes mentioned previously. Consequently, it is even more difficult to conserve the imbedded ice in regions where human activity causes changes in the regime of the ground water or otherwise disturbs the natural conditions of the region. The probability of conserving imbedded ice beneath buildings is generally slight. In some cases, imbedded ice can be conserved beneath earth structures, particularly when the ice occurs at a depth of 5 to 6 m below the ground surface.

It is of great practical importance to determine the type of ice mass occurring in the ground, since suitable measures for avoiding or minimizing the harmful effect of the ice on the stability of structures depend on the origin of the ice as well as on its horizontal and vertical dimensions.

Icings (the Yakut term is "taryns") constitute one of the most widespread and typical hydrological phenomena of the permafrost region. In accordance with location and origin, icings may be classified into the following three categories: river, ground, and spring icings.

River icings are formed by river water as follows. As the winter freezing of a river sets in, the effective channel of the river gradually becomes constricted because of the ice cover and the freezing of the banks and eventually becomes too small for the entire volume of flowing water to pass through it. Since the water cannot pass through the alluvium, it overflows the banks or the river valley or forces its way to the surface through the river ice. As the water spreads out over the ice and the banks, it freezes in successive sheets and forms relatively extensive and thick stratified layers of ice over the river ice or in the valley. The extent and thickness of this ice is quite

variable; it may cover an area of several hundred square kilometers and commonly attains a thickness of several meters. This icing originates either in the river channel or in the floodplain. If the icing is not too large and originates in the regular channel of the river, it is usually confined within this channel and only seldom fills it and spreads into the valley. Most extensive icings occur in marshy streams. They recur every year in approximately the same places; although their locations change, the change is very gradual. The banks of small streams lack vegetation and are covered with stones at the points where icings occur; the stream flows through numerous channels in the stones. Figure 5 is a representative picture of a river icing; it shows the summit of an icing mound on a small stream. An icing on a river in Yakutia is shown in Fig. 54.

Icing fields melt slowly; some vanish at the end of spring, others vanish during the summer, and a few remain until the following year. Some icings in river channels or valleys contain round or oval ice mounds with cracked summits; occasionally water flows from these cracks, increasing the size of the mound upon freezing. One cause of mound formation in a channel is that the unfrozen water accumulates under the ice, uplifts its, and forms a swelling; another cause is that the water flowing onto the surface of the ice freezes near the place of emergence. The mound gradually grows until a crack appears in it; then the water bursts violently to the surface, shattering and scattering the ice. The crack forms suddenly, and the shattering of the mound ice resembles an explosion. Icings readily deform structures located in their vicinity and, consequently, constitute a hazard to all types of structures.

The slab from a river icing in Yakutia, shown in Fig. 6, illustrates the phenomena accompanying the explosion of a river-icing mound. According to V. G. Petrov [3], this icing contained six large mounds. After the explosion, the water poured out in a stream 75 m wide and advanced a distance of about 5 km. This water carried the slab shown in Fig. 6, as well as several other slabs of ice, a distance of 120 m. The largest slab measured 19 by 5 by 2 m.

Ground icings are another variety of icing; their origin is usually in the suprapermafrost water. The formation of suprapermafrost or ground icings occurs as follows. As winter sets in, the level of the suprapermafrost water begins to lower gradually, since the surface water vanishes because of the cold and ceases feeding the suprapermafrost water. This lowering continues until a constant level is established. As the upper ground layers freeze, an impervious crust is formed and interferes with free circulation of the suprapermafrost water within the active layer. As individual sections of ground freeze ever deeper in the course of continuously increasing cold, internal pressure develops and the water under pressure seeks an outlet to the ground surface. If the pressure is large, the water bursts through the overlying frozen layer, often in several places, spills over the surface and forms an icing which occasionally covers a considerable area. Figure 7 shows such an icing near a railroad; the picture was taken on May 28. The icing melts very slowly, vanishing only in the middle or at the end of summer. As a result, trees perish and other vegetation withers at the sites of icings. The area covered by ground icings measures tens, hundreds, infrequently thousands of square meters; hence, it covers considerably less than that of river icings.

As the cold increases, the small outlets for the suprapermafrost water become frozen, while the remaining outlets occur in the form of occasional mounds usually located near bushes or tree trunks. Such mounds are like small volcanoes erupting water. They increase in size in the course of time, but do not become very large (Fig. 8) because the active layer soon freezes completely and the entire suprapermafrost water is converted into ice.

When the active layer is underlain by water-bearing talik resting on rock or on permafrost (Fig. 1), or when the suprapermafrost water is under high pressure, the accumulated water causes swelling of the ground and gradual formation of a mound if the overlying frozen layer is sufficiently firm and elastic. During this process, the upper frozen layer separates from the water-bearing talik and rises a certain distance together with everything located on its surface (Fig. 9). Inside the mound there is ice, or ice mixed with water. The mound increases

in size in the course of time; cracks from which water flows form on its crown and along its sides. This water flows over the snow and ground and freezes in successive ice sheets that cover a certain area. If trees grow at the site where the mound is formed, they rise with the mound and assume an inclined position. The ice melts during spring and summer, and the mound settles. The mounds attain a height of 4 to 5 m. Figure 10 is a schematic diagram showing the formation of a mound of ground icing.

In accordance with the preceding, the water which forms ground icings (that is, the mound ice within the ground and on its surface) does not originate at the site of the mound but flows to it from all sides, as in the case of river icings. As the frosts continue and the active layer freezes deeper, the hydrostatic pressure in the ground, which produced the icing mound, increases to such an extent that the ground water is forced to the surface through gradual seepage from cracks in the frozen crust of the mound or through sudden explosion of this crust and subsequent discharge of large quantities of water. Such explosions are accompanied by considerable noise, and their force is so great that the frozen ground and ice are shattered into huge chunks of which some are scattered in all directions and some are carried for a considerable distance by the water streams. The explosion of a mound is capable of destroying nearby structures.

Figure 11 shows the summit of a mound of ground icing. The crack in this mound is more than 1 m deep. The picture was taken early in the spring when the mound had already settled somewhat, but an ice lens still remained in the ground. The ice cover on the mound and the ice on the walls of the crack had not melted. The inclined trunks of the trees are readily visible.

When a mound is cut open, it is found that either an arched or a shallowly convex ice lens has been formed under the surface layer of frozen ground; either one attains a thickness of about 1 m. At the bottom of the mound (under the ice) there usually occurs a viscous, semi-liquid mass about 1 m thick and underlain by the permafrost. Ground icings, like river icings, tend to recur every winter in the same places,

although there are numerous exceptions to this rule. Of particular interest are the mounds known in the Yakutia as "boolgoonyakh." These mounds attain a height exceeding 10 m and contain masses of ice. Upon formation, the boolgoonyakh slowly grows for several years; then a crack occurs in its summit, and it gradually vanishes in the course of several years. Its considerable size and duration distinguish the boolgoonyakh from an ordinary ground icing. Boolgoonyakhs occur widely; they are found in northern Yakutia at the latitude of Yakutsk proper, in the former Amur Province, in the Transbaikal region, in the valley of the Upper Angara River, and on the Yalmal Peninsula. Their origin has not yet been definitely established.

In many cases, icings are caused by subpermafrost water; such icings are designated as spring icings. Externally, spring icings are quite similar to ground icings. Spring icings form when the subpermafrost water emerges and freezes at the surface. The temperature of these springs is sufficient to thaw a passage for the water through the permafrost and to prevent winter freezing in the active layer. As the water emerges at the surface, it flows down the valley slope in a relatively shallow stream and freezes in successive sheets of ice. It should be noted that the outlets of these springs occur in the winter at different places than in the summer; stone outcrops or moss cover make it difficult to determine the exact location of the summer outlet. In the winter, however, the outlet usually is gradually displaced up the slope, which is readily evident from the icing formations and its location becomes constant only when intense frost sets in. Determination of the exact location of the spring outlets is of major practical importance in proper planning of structures. The temperature of such springs is fairly constant at 0.5° to 3° C.

Spring icings appear in December, but sometimes they do not appear until the end of January when intense frosts finally freeze the corresponding flow in the active layer. Growth of the icing begins immediately after formation, continues throughout the season, and ends at the end of April or the beginning of May. During the entire period of growth, these icings readily manifest that flow occurs beneath the

surface crust of ice; the water bulges this crust in the form of mounds or bursts through the crust and flows on its surface. The extent of spring icings is most intense at the end of winter (in March and April) when the frosts become milder. This aspect constitutes the major difference between spring icings and ground icings, as growth of the latter ceases entirely in the middle of winter.

Spring icings begin to form at a different time from ground icings; the latter occur at the beginning of winter, while the former do not occur earlier than December, January, or even February. Ground icings cover considerable areas. According to geologist A.V. Lvov, for example, a spring located on the right bank of the Black Uryum River, roughly opposite the mouth of the Yarnichnaya tributary, produced an icing covering 70,000 sq m. Figure 12 shows a spring icing progressing towards the river. Spring icings, like ground icings, cause considerable difficulty in road maintenance, and may damage or even destroy structures.

Ground icings also include so-called icefalls, that is, icings resulting from freezing of ground water emerging to the surface on sheer cliffs, on steep river banks, or on the sides of railroad cuts where they are extensive and require major efforts to remove them or to prevent their recurrence. Figure 13 shows an icefall in a railroad cut.

The morphological phenomena of the permafrost region include peat mounds and tundra polygons. Sumgin classifies ~~both these~~ topographical phenomena as icing mounds. He found that the vertical section through a peat mound in summer is as follows: an upper layer of thawed peat not more than 70 to 80 cm thick (depending on the time of the year and the latitude), a layer of frozen peat, frozen ground, and finally (occasionally) an ice core. Tundra polygons are small, round, or oval areas free of vegetation and differing radically from the surrounding tundra that is covered with plant life. The textural constituents of the polygon ground are usually segregated so that the coarse material accumulates near the edges of the polygon, occasionally forming a stone boundary. Peat mounds and tundra polygons are such typical phenomena that marshes containing mounds are termed "mound marshes," and tundra containing polygons

is termed "spotted tundra."

Mounds similar to peat mounds occur in marshy places in the Far East, but their height does not exceed 50 to 75 cm. There, the areas covered with such mounds are called "graveyards." According to Sumgin, such landscape phenomena are due to (1) pressure in the taliks, resulting from adfreezing between active layer and permafrost under certain conditions, (2) hydrostatic pressure of the water, and (3) direct expansion of the freezing water which is forced into the mound during its formation. In some cases, slud is involved in place of water. Figure 14 shows a ruptured frost mound; the "drunken forest," resulting from ground swelling, is clearly seen.

In the permafrost regions of the Far East there are extensive marshy areas which are locally known as "mari."* The nature and properties of mari are quite different from those of common bogs. Mari are relatively shallow, rarely exceeding 2 to 2.5 m in depth, have a thick moss cover, contain primarily moss peat and sedge peat, while almost entirely lacking wood peat, and occur not only in floodplains, lowlands and level areas, but even high on the slopes of mountains.

The nature of mari is essentially as follows. Impermeability of the permafrost and occurrence of natural rock formations at shallow depth in regions of abundant precipitation during a brief time interval, as well as late thawing of the ground when spring sets in rapidly, create conditions favorable to the development of swamp plants. These plants, moss and sedge, grow rapidly and become firmly rooted in the upper thawed and wet ground layers to which moisture is supplied by melting snow, rains, and condensation of water vapor. In the course of time, this vegetation itself becomes a factor retarding water run-off and thus intensifying the swampiness. These factors are so effective that even very steep slopes often tend to become boggy and assume the aspect of swamps. Ample moisture and low temperatures kill

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* Plural of mar. - M. P.

the vegetation and simultaneously retard the decomposition of the organic matter, forming a thick layer of peat consisting of moss and other swamp plants. This layer is highly hygroscopic so that it is always quite wet.

The thick moss and peat covers on mari have a marked negative effect on heat balance of the underlying ground layers. During the summer, the saturated moss and underlying peat layer consume large quantities of heat in the process of evaporation while retarding heat penetration into the ground because of their low heat conductivity in thawed state. During the winter, the saturated moss and peat become a solidly frozen mass so that their heat conductivity is nearly four times as large as during the summer. As a result, the upper limit of permafrost beneath mari often occurs at a depth of 0.20 to 0.50 m. Figure 15 (left) shows a layer of ice beneath a relatively thin cover of moss and peat.

There are two types of mari mossy (smooth) and hummocky. Most mari are of the second type. The hummocks are densely distributed, attain a height of 0.75 to 1.00 m, and greatly impede runoff; since the absorptive capacity of moss and peat is large, this impedance further facilitates saturation of the mar. The mar shown in Fig. 15 is of the hummocky type, but the relatively small hummocks are nearly invisible because of the extensive sedge. Figure 16 shows a distinct hummocky mar. The hummocks consist of moss and peat and often contain a permafrost core.

Practice proves that mari can be drained. Their melioration occurs in this manner. As the water table is lowered, the upper layer dries out and the dry moss is artificially burned, leaving a large quantity of ash. The mar surface, black and dry after the burning operation, is readily warmed by the sun so that further and intense drying occurs. Consequently, the type of vegetation changes, summer thawing penetrates deeper, the permafrost recedes, and the moss is replaced by meadow grass, brushwood, and even trees. The mar is transformed into a meadow. These considerations imply that it is sometimes expedient to carry out at least elementary drainage operations for removing surface water and some ground water from mari.

It should be noted that there are two additional phenomena directly related to permafrost. They are: solifluction and asymmetry

of the latitudinal valleys (in the Far East). The presence of permafrost affects the characteristic topographical forms on a much larger scale. The effects include solifluction, which is defined as slow gravitational flowing of masses of superficial material over the upper limit of permafrost; such flow occurs even in the case of very shallow slopes. Entire terraces, resulting from such motion, were discovered in the Far East by S. P. Kachurin.

Explorers should be careful not to confuse these unstable solifluction terraces (Kachurin uses the term pseudo-terraces) with real erosion terraces. It has been definitely established that the opposite sides of latitudinal valleys in the permafrost region, primarily in the southern areas, have unequal slopes. The slopes facing north are shallow, while those facing south are steep. This is explained by the fact that the former slopes are stabilized by the permafrost which occurs at shallow depth and retards the scouring effect of erosion; in addition, spring thawing of the surface layers on these slopes proceeds slowly because they receive a relatively small amount of heat, so that the resulting streams of water are relatively weak. The opposite slopes, however, particularly in southern permafrost areas, are exposed to rays of the high summer sun which impinge on the steep slopes almost at right angles and heat them intensely. Therefore, the permafrost on these slopes occurs at a greater depth, and the active layer thaws earlier and more rapidly than on the slopes facing north. The result is that during the second half of the summer, during the rainy period, powerful erosive streams flow down these slopes, washing away the alluvium and diluvium and often exposing the bedrock that would form a steeper slope.

In the case of longitudinal valleys, the opposite sides usually have similar slopes and, other conditions being equal, their permafrost occurs at the same depth. This difference in steepness and nature of the slopes is of considerable importance in railroad construction, as it affects location surveys for tracks and various structural sites.

Bouldery alluvial and diluvial talus frequently occur in permafrost regions. Abrupt fluctuations of atmospheric temperature and the large amplitude of these fluctuations in the permafrost region of the Far East, particularly in the southern latitudes, cause intensive mechanical weathering of rocks. The widespread occurrence of such phenomena in this area is due also to the fact that the active layer tends to force the stones out to the surface upon freezing. Constituting a type of alluvium, these talus usually comprise fairly large rocks and occur on watersheds, mountain passes, terraces, shallow slopes, and occasionally in valleys. Therefore, they are fairly stable, that is, they do not tend to move, except in cases when they occur on slopes and the spaces among the boulders are filled with fine-textured ground. Under such conditions, they may undergo downhill motion of solifluction type. In addition, considerable flow of suprapermafrost water over the underlying bedrock, as well as outlets for subpermafrost water, may occur in both types of talus. As stated previously, this causes large icings at these places. Figure 17 shows a bouldery alluvial talus on a fairly level area near a road.

Figure 18 shows a bouldery alluvial talus covered with scrub and located on a slope near the Niman River. Partial excavation showed that the depth of the talus ranged from 2.5 to 4.0 m. The underlying bedrock cropped out to the surface in some places in the form of buttes. This talus consists of angular boulders averaging 0.25 to 0.60 cu m. Individual boulders had volumes of 1 to 2 cu m. The top boulders were loose and could be readily moved. Below the top layer, beginning at a depth of about 1 m, the spaces among the boulders were filled with gravel and rubble.

In many cases, the alluvial talus are covered with a carpet-like layer of moss 20 to 30 cm thick. Figure 19 shows a moss-covered talus on a slope near the Niman River. Walking on the talus caused some boulders to sway or become displaced in spite of the moss cover.

Talus of diluvial origin occur on slopes, in avalanches, and at the feet of slopes, often in the form of huge cones. The material in these cones is relatively segregated with respect to size, the largest boulders occurring at the bottom, while the remaining material

becomes increasingly fine towards the top of the cone. There are two types of diluvial talus. Dead talus, which no longer receives any replenishing material and has ceased moving, and live talus which continues to increase in size and to move downhill. It should be noted that road building or other construction operations on dead talus may disturb the established equilibrium and transform this talus into a live one, that is, the talus may begin to move again. Alluvial talus overlie the bedrock from which they originate, while diluvial talus may cover alluvial rocks.

The proceding demonstrates the practical importance of thorough investigation of alluvial and diluvial talus for proper utilization of construction sites and for timely application of preventive measures to assure structural stability.

Rivers in permafrost regions have several characteristics requiring particular attention. The presence of permafrost and the freezing of the entire suprapermafrost layer, which occurs in most cases, cause a marked decrease in the winter supply of ground water to the rivers. Consequently, the rivers are at very low stages in the winter. Wherever the winter temperature is quite low and the snowfall during the first half of winter is slight, the rivers freeze clear through and the water flows in the winter through the porous alluvium. However, this alluvium freezes partly or entirely; in the latter case, the river ceases flowing in the winter. Such complete freezing of the entire stream in local areas causes formation of the river icings described previously. Figure 20 shows a river frozen solid during the winter.

Rivers show a prominent decrease in stage during the winter. In some cases, after an ice layer has been formed, an air space is formed beneath the ice when the stage decreases further. Subsequent freezing of the water forms another ice layer. Thus, several such ice layers may be formed. Even as large a river as the Zeya River, a tributary of the Amur River, may freeze solid. M. Ya. Chernyshov reports that the discharge of the Zeya River in the winter is 1000 times smaller than in the summer. In several places on the Gilyui River, where the water does not freeze in the winter, the water

becomes polluted and begins to stagnate.

A most important characteristic of rivers in northern and eastern Siberia is formation of bottom ice. Such ice forms very rapidly in large quantities and is capable of damaging river structures. Bottom ice constitutes a serious obstacle to direct intake of river water for water supply purposes. Bottom ice usually forms on sandbanks where the flow is retarded, along the shore, and in deep places near stones on the bed. Having accumulated to some extent, the bottom ice rises to the surface and is carried by the flow; it forms a fairly heavy ice flow in the autumn. This causes obstructions and ice jams so that the stage in many rivers rises 3 to 5 m above low stage during the breakup of the bottom ice.

The obstructions and ice jams may damage river structures, while the resulting rise in stage causes flooding of the surrounding area. It is obvious that the river regime under such conditions needs careful investigation since lack of proper data may result in structures erected on unsuitable sites.

In regions where the snow cover is negligible, which is the case in a large part of the permafrost zone (particularly in the south latitudes between 90° and 100° east longitude), the rise in river stage during the spring is quite small. Ice flows practically do not occur. Summer floods, on the other hand, are characterized by a rapid rise in stage, high water level, high velocities, and a rapid drop to low stage. This is primarily due to the torrential nature of the summer rains and the mountainous topography comprising steep water-collecting depressions and valley slopes. In addition, supersaturation of the active layer, typical of permafrost areas, has some effect on the nature of such flood states; where moss cover is lacking, the active layer absorbs only a relatively small proportion of the torrential precipitations.

Because of extremely high flow velocities during floodstage, many rivers carry trees, silt, gravel, and pebbles, and roll even large rocks. Consequently, the debris rapidly forms a dam at the protective timber ice-deflectors near a bridge or culvert. This may cause a rise in water level that will endanger the structures. Near a certain railroad

station, for example, such a flood covered the entire right-of-way with 1 m of water, deposited about 100,000 cu m of silt and gravel on the tracks and surrounding areas, and washed away the roadbed at the approaches to a bridge across the river. It should be noted that trees in the permafrost region are readily uprooted by wind or water streams because they cannot root deeply in the ground. There are instances on record when rivers have risen several meters above low stage during spring floods. The Niman River has been known to rise 3 m. Figure 21 is a view of the Niman River at low stage. The limit of normal rise in stage is clearly visible in the photograph; it occurs at vegetation level along the shore. An occasional rise in water level would extend into the vegetation zone as well.

Lakes are a common occurrence in the permafrost regions. They occur mainly in valleys and constitute the remains of old rivers. In most cases these lakes are isolated basins without outlets that are fed by surface runoff and occasionally by alluvial water. Nearly all lakes freeze solid in the winter, with the exception of those basins that are extremely deep or are fed by warm subpermafrost water.

Thermokarst lakes, referred to previously, are a special category of lakes occurring in large numbers in the region of the Zeya, Bureya, Nara, and Selemdsha Rivers. Thermokarst lakes are shallow and small. They are fed exclusively by surface runoff and suprapermafrost water. Their water capacity is limited, and they cannot be readily utilized as sources of water supply.

With regard to the permafrost layer as a structural base, the following need be stated. As mentioned previously, permafrost is not a constant and unchanging phenomenon. On the contrary, the permafrost layer undergoes continuous changes; its extent and thickness are affected by numerous factors. Analysis of the aspects of imbedded ice, for instance, has shown that disturbing the natural state of the locality may affect conservation of this ice; in the description of mari, in addition, it was noted that melioration of mari results in considerable lowering of the upper permafrost limit. It is quite evident that the construction engineer must not ignore the fact that both the position

and state of the permafrost layer are subject to change.

When frozen ground thaws, it settles and its bearing capacity tends to diminish. This results in considerable settling of the structures erected on this ground; moreover, settling is usually not uniform with respect to the perimeter of a given structure, so that structural deformations vary. When thawing is not uniform, which is commonly the case, the thawed ground tends to slide over the frozen ground. The sliding surfaces produce stresses in the ground; these act horizontally on the foundations and occasionally cause creeping or shearing of the entire structure or parts of it.

It is generally feasible to conserve the permafrost near and beneath a structure, but it requires special heat-insulation measures. Moreover, the upper permafrost limit can be artificially elevated in some cases by use of proper thermotechnical methods. It is accepted that conservation of permafrost is generally possible when its temperature is sufficiently low, not above -0.5° C. It is doubtful whether conservation of permafrost is possible at higher temperatures. If the permafrost in a given area undergoes degradation, no attempts should be made to conserve it.

Thawing of the permafrost layer beneath a structure is due to numerous factors; the major ones are discussed here. The level of the upper permafrost limit is affected mainly by established regime of the ground and suprapermafrost waters. Drainage of the surface layer, by means of ditches, for example, lowers this limit. The magnitude of this lowering is indeterminate and cannot be evaluated in advance. Even when no direct drainage measures are used, the very erection of structures may disturb the existing quantity and direction of ground-water flow and thereby produce a change in the permafrost layer.

Vegetation cover has a marked effect on the regime of ground water. Removal of trees, scrub, and grass or moss cover has a direct effect on the permafrost layer in addition to its effect on the regime of ground water as it disturbs the normal, established temperature of the ground in the given area. Construction operations usually cause destruction of the vegetation cover and, consequently, alter the upper

permafrost limit. Finally, the structures themselves, being extended into the ground, may constitute heat conductors and thus disturb the state of the permafrost. Many structures are heated, while others constitute appreciable sources of heat because of the technological processes occuring in them. Part of this heat is frequently transferred to the ground. In any case, heating of the ground near and beneath the structure causes thawing of the permafrost. This thawing can be estimated to some extent by means of proper thermodynamic calculations; it should be noted, however, that such an estimate would be approximate and would give only a general idea of the probable extent of thawing of the ground.

Many types of permafrost cannot support any load when thawed and settle considerably upon passing from frozen to thawed state even at relatively low moisture content. These types comprise loamy and particularly silty grounds. Sand and gravelly grounds, which drain well, settle less. Gravelly and sandy grounds with a moisture content up to 30 per cent by weight settle little and the effect on structures is minimal.

In selecting a site for a given structure, it is essential to analyze the ground and determine its moisture content since structural stability largely depends upon the nature of the ground. In accordance with the preceding, ground that drains well and has a low moisture content is preferable as a construction site. Examination of the moisture diagram of Fig. 3 reveals that cases may occur in which the ground near the upper permafrost limit is supersaturated, while the ground at a depth of 1 to 2 m below this limit contains little moisture. It is obvious that structural foundations should be extended below the upper permafrost limit in such cases, so as to utilize the bearing capacity of the deeper but drier permafrost layer. Layered permafrost is seldom suitable as a structural base; it is best to dig through it and support the structure on a thawed layer or on the solid permafrost.

The aspect of vegetation in the permafrost regions requires special consideration. The presence of permafrost in the Far East,

in conjunction with the climatic conditions in that region, causes widespread swamps and the occurrence of sparse woods of predominantly larch trees. The larch is better adjusted to permafrost than other trees because its root system is surficial and does not penetrate deep into the ground. Nevertheless, even the larch rarely attains normal size, while in many cases, particularly on mari, it looks unhealthy and stunted. The pine, which has a different type of root system, occurs rarely and in isolated clumps. It occurs mainly on bedrock covered with a crust of alluvium or gravel, on layers of detritus and talus that drain well, and on sunny slopes, where there is no permafrost or it occurs fairly deep. Fir is more common than pine because its root systems is similar to that of the larch. The occurrence of birch is another indication that the permafrost is not far below the surface; under these conditions, the birch dies young and its wood rots while the tree is still standing.

Large numbers of trees felled by the wind (Fig. 22) likewise indicate that the upper permafrost limit occurs at shallow depth, since the tree roots cannot penetrate deep into the ground.

Accordingly, the nature of tree growth makes it possible to determine whether or not permafrost is present and, in many cases, the approximate depth of its upper limit. This gives the surveyor, designer, and construction engineer an opportunity for preliminary orientation of a given structural area on permafrost.

In order better to comprehend the problems discussed here, the following material is appended; it comprises selected standards and technical conditions for design of foundations and footings on permafrost and contains numerous interesting and valuable data:

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II. GENERAL ASPECTS

2. Permafrozen ground or permafrost is defined as ground with negative temperature and not subject to seasonal thawing.

3. The ground layer which freezes in winter and thaws in summer is called active layer.

4. Permafrost is termed merging if its upper limit merges with the active layer; it is termed nonmerging if an unfrozen layer separates this limit from the active layer. Dry ground (rock, dry gravel,

dry sand, etc.) which is not adfrozen and has a negative temperature is termed dry permafrost.

5. The thickness of the active layer is determined by the maximum depth of thawing in the case of merging permafrost, and by the maximum depth of freezing in the case of nonmerging permafrost.

6. In vertical section, permafrost may be continuous or layered, that is, consisting of successive layers of frozen and thawed ground.

7. In the horizontal plane, permafrost may have the following distribution: (a) continuous, (b) containing islands of thawed ground (taliks), and (c) in the form of islands or separate lenses.

8. The temperature of permafrost ranges from 0° C to -3° C in its southern regions, and reaches -7° C or even lower in the northern regions. The upper layers of permafrost have a temperature near 0° C, but it may drop during winter freezing.

9. Three types of subsurface water occur in continuous permafrost: (a) suprapermafrost water, which occurs within the active layer or in the talik separating this layer from the permafrost, (b) intrapermafrost water, which circulates within the permafrost proper, and (c) subpermafrost water, which occur beneath the lower permafrost limit. It should be noted that all three types of water usually are in contact with each other and under pressure.

10. The permeability of fine-textured frozen ground is practically zero; coarse-textured ground, on the other hand, is relatively permeable at low moisture content (saturation with ice).

11. The following phenomena accompany freezing of the active layer:

a. The active layer may adfreeze to the footings in the process of freezing. In the case of swelling ground, the active layer increases in volume and tends to heave the foundations to which it has adfrozen. This may cause rupture of the foundations, formation of empty spaces beneath them, or sliding of the frozen ground along the foundations surfaces.

b. When the foundation base is in the active layer, the heaving forces beneath this base may raise the entire foundation.

c. The active layer, expanding as it swells, produces not only uplift pressures, but also horizontal pressures which cause structural deformation.

d. In the process of freezing, ground and surface water (rivers and lakes) may become locked and compressed between an overlying layer of frozen ground or ice and the underlying permafrost or other impervious layer. The resulting hydrostatic and hydrodynamic pressures may either uplift the upper frozen crust simultaneously with the foundations adfrozen to it, or may cause the compressed water to burst to the surface or into buildings, so that cracks form in the frozen crust (icing phenomena) or the pressure increases on the exterior foundation surfaces.

12. The following phenomena accompany thawing of the active layer:

a. Foundations that were heaved during the winter do not necessarily assume their former position because of friction and adfreezing between foundation and ground, or because the spaces beneath the footings or in places where the foundation was ruptured are filled with ground or ice.

b. Thawing of the ground proceeds more rapidly near wall footings facing south, so that the ground water tends to accumulate there and turn the ground into fluid.

c. The thawed ground forms a slippery and moist surface which facilitates the occurrence of landslides and creep.

13. The following phenomena accompany thawing of the permafrost:

a. The regime of surface and ground water changes, water passages are formed between various levels, ground water appears and vanishes, etc.

b. Local sags occur in supersaturated ground, and thermokarst lakes form in places where thick layers of ice occur in the ground.

c. Sloping surfaces, which cause sliding of foundations, occur as a result of nonuniform thawing.

d. The bearing capacity of friable ground is decreased, particularly in the case of fine-textured ground.

CHAPTER II

DEFORMATION OF ENGINEERING STRUCTURES UNDER PERMAFROST CONDITIONS

A. General Considerations

Numerous structures of various dimensions, design, and functions, erected in the permafrost region, undergo continuous deformation and even destruction, causing waste of considerable labor and material. In some places the deformations are so extensive and common that it becomes doubtful whether it is at all feasible to erect structures that would not deform. In reality, such is not the case; although the deformations are extensive and common, they are not necessarity inevitable. In many cases the deformations are less attributable to particular local conditions than to the facts that the engineers are inadequately informed and trained for work in this area and that the unique local conditions are not taken into consideration. Experience proves that construction on

permafrost is feasible, but it requires knowledge of permafrost and its phenomena and the ability to evaluate local factors.

Structural safety can generally be assured if the construction site is well chosen, the design and layout are correspondingly suitable, the measures for preventing structural deformations are proper, and the construction operations are appropriately based on local permafrost conditions and carried out with adequate skill. Examination of specific instances of structural deformations and related factors help to detect the causes involved, to determine the design and methods that would assure structural stability under given conditions, and to clarify the aspect of selecting a construction site.

Basically, the major causes of deformation are heaving and settling of the entire structure or any part of it. Both heaving and settling occur suddenly and nonuniformly. Settling or heaving causes the structure to tilt, buckle, warp, and develop fairly large cracks. Once settling starts, it does not stop soon and usually causes considerable damage. Heaving recurs every winter so that the structure may finally be put out of commission. Settling results from thawing of the upper portion of the permafrost layer upon which the structure is based. Swelling results from freezing of the upper, wet ground layer which has been designated previously as the active layer. In the process of swelling, the wet ground is displaced and displaces the structural supports adfrozen to it.

Structures are deformed in some cases because of icings formed near or beneath them. The icings may flood a given structure and fill it with ice; in addition, they may mechanically damage the structure during explosion of an icing mound. Deformation of earth structures requires special consideration. These structures are affected by swelling, settling, icing action, extension of freezing into the core, and slippage due to excessive moisture in the ground.

The following paragraphs deal with the basic types of deformation to determine their nature, magnitude, causes, and accompanying factors.

B. Structural Deformation Due to Swelling of Active Layer

1. General Remarks

The active layer is exceedingly wet, as shown in Chapter I, often even supersaturated, and swells intensely during winter freezing. Swelling usually affects large areas. It is usually assumed that clayey and loamy ground are the types of ground that would swell readily and intensely under ordinary conditions outside the permafrost region. Under permafrost conditions, however, all types of ground tend to swell when the moisture content is high and when there is inflow of suprapermafrost water. This is due to low atmospheric temperature, which may drop to -40° C or even -50° C, and the fact that the upper ground layer is underlain by hard rock or permafrost. Moreover, even gravel and coarse sand, particularly if they are not quite clean and contain silt, tend to swell under highly unfavorable conditions.

The theory of ground swelling is discussed in the works of M. I. Sumgin, Professor Voyslav, and S. Taber, so that it is unnecessary to discuss it in detail here. The present work deals only with the basic aspects of this phenomenon that pertain to its nature and are essential for the study of structural deformation. When a given ground layer freezes under natural conditions, ice crystals are formed in the layer if it is sufficiently wet. These crystals increase the volume of the ground, causing swelling and raising the ground surface. In the process of swelling or expansion of ground volume during freezing under natural conditions, the water involved comprises both the water contained in this volume and in most cases, a considerable quantity of inflowing water. This inflow increases the volume of the swelling ground far beyond the normal 9 per cent increase in volume due to freezing (transformation into ice) of the water contained in the ground. Professor Voyslav succeeded in obtaining a volume increase of 140 per cent of the original; this occurred under laboratory conditions of unlimited capillary inflow. The American researcher S. Taber states that he observed an increase of 60 per cent in ground volume at a given point.

Inflow of water into a freezing ground layer is due to capillary rise, force of crystallization of ice (that is, stresses occurring

in the ground during freezing), and ground-water pressure caused by either freezing or local conditions. The causes of water-inflow are of practical importance since they determine the nature of the measures for preventing the inflow and, consequently, reducing or eliminating swelling. The rise of water in frozen ground is not improbable; numerous tests and investigations have definitely established that water in fine capillaries resists freezing at temperatures considerably lower than those occurring in the ground under the conditions involved. The increase in volume when water is converted into ice occurs instantaneously, so that swelling occurs intermittently.

The preceding analysis relates the phenomenon of ground swelling to the quantity of water contained in the ground and to the water which flows into the freezing ground from the outside, but not to the nature of the ground itself. In actuality, ground that drains well usually contains less moisture and its capillarity is smaller. Consequently, swelling of such ground occurs less frequently and its magnitude is smaller, although it is sufficiently large to cause deformation of numerous structures.

Observations show that the factors causing intensification of heaving of structures are as follows, other conditions being equal: (1) increased content of loam and silt in the ground, (2) lower temperature, (3) low compressibility of the ground beneath the active layer, (4) increase in thickness of the swelling layer within certain limits, and, occasionally, (5) increase in rate of freezing of the ground. Experience has proved that loamy and silty grounds swell most, while gravelly and pebbly grounds swell least.

The rate of freezing of fully saturated ground is generally the same for all types of ground; under natural conditions, however, this rate depends on the state of the surface cover, since this cover determines the rate of flow of cold into the ground. In addition, the rate of freezing of the ground tends to increase when structural footings are placed in the ground, since they act as conductors of cold.

All types of permafrost have low compressibility; accordingly, swelling is particularly pronounced whenever the active layer rests

directly on top of the permafrost. The same is valid in the case of the active layer underlain by rock.

Although the active layer constitutes the swelling layer, yet its entire thickness does not participate equally in the heaving of structures. Ordinarily, the uppermost portion of the active layer does not swell, while the lowest layers swell very little, since they do not contain the necessary amount of water because of freezing of the suprapermafrost water, and their temperature does not drop as low as that of the upper layers. On the other hand, some thin ground layer participates in the process of swelling at any given moment. K. D. Morozov [3, pages 48-49] points out that ground layers which already are frozen and have been subjected to swelling tend to swell little or not at all. This has recently been confirmed by test conducted by N. I. Bikov at a permafrost station; he found that the swelling layer did not exceed 1.5 m in an active layer 2.0 to 2.5 m thick.

Basically, swelling of the ground occurs as follows. As the temperature of the air drops below 0° C, the ground begins gradually to cool; this cooling of a given freezing layer varies with distance from the ground surface. Most of the water in the ground is converted into ice immediately, while a small portion is supercooled below 0° C without freezing. Conversion of the water into ice causes the ground to increase in volume, that is, to swell. Due to inflow of water from underlying, as yet unfrozen ground layers, a certain quantity of new water reaches the frozen ground layer. This water penetrates into the fine fissures of the frozen mass, combines with the supercooled water there, and, becoming supercooled itself, freezes partially and envelopes the ice crystals that had previously formed in the ground, facilitating their growth and causing further swelling. Part of the water, however, remains supercooled. Thereafter, another quantity of water reaches the frozen and freezing layers, and the process recurs until there is no longer any inflow of water; swelling ceases then.

This description simplifies the process of swelling to some extent, since in actuality the inflow of water occurs not intermittently but relatively continuously. However, this does not change the major

aspect of the problem involved. The supercooled water is instantaneously converted into ice, so that the swelling is intermittent rather than gradual. Simultaneous freezing of a large quantity of supercooled water is due to the fact that supercooling of water under given conditions occurs only to a definite degree, after which the water must convert into ice. On the other hand, this conversion is aided by the stresses, developed in the frozen mass as a result of lowered temperature, which cause deformations that shake the frozen crust and agitate the supercooled water in the fine fissures of the ground. Seismologists have recorded the oscillatory deformations occurring in the ground during freezing in the fall [4].

In accordance with the preceding, it should be assumed that only one, relatively thin, truly effective layer which swells at a given instant can affect a structure at that time [3, page 18]. Available data indicate that the thickness of this effective active layer under natural conditions increases (from the surface downward) during freezing of the active layer, until freezing reaches a certain depth; having attained a definite maximum, this thickness decreases as freezing progresses deeper. Accordingly, the process of swelling is increasingly rapid at first, reaches a certain maximum rate, and then slows down. Unfortunately, the factors influencing the freezing of the active layer under natural conditions have not been adequately investigated. Therefore, the rate of ground freezing, its dynamics, and other important aspects of the phenomenon are not known as yet.

In order to study the phenomenon of heaving of a structure during freezing of the active layer, it is expedient to investigate the heaving of a pile of weight Q, under an external load P, and extending a distance ΣH into the ground, as shown in Fig. 23. The phenomenon occurs as follows. For convenience of analysis the entire process is divided into individual stages occuring within corresponding time intervals, and it is assumed that a definite layer freezes during a given time interval, at first h, then h_1, then h_2, etc. When the initial frosts set in, the upper ground layer h firmly adfreezes to the pile. Freezing of this layer causes swelling because its original

water and the water that reached it from the outside are converted into ice. The swollen ground, which is adfrozen to the pile, tends to uplift the pile until it assumes position 2 (Fig. 23). This is resisted by friction T between the surface of the pile and the thawed ground underlying the frozen layer, the weight of the pile Q, and the external load P. The heaving force N is transmitted to the pile through the surface of adfreezing between the pile and the swelling layer. The force N must overcome the sum of the forces $\sum S = T + Q + P$.

If the bond between the frozen ground and the pile is insufficiently strong, so that the total adfreezing strength of the given ground layer is not greater than $\sum S$, the adfrozen layer will separate from the pile and assume position 3 (Fig. 23), and the surface near the pile will rise a certain height Δh with respect to the initial height; a gap is formed around the pile. Pressure (Fig. 4) forces water from the lower, unfrozen layers into the gap and the frozen layer. Progressive freezing in depth causes freezing of a new layer which adfreezes to the pile. Simultaneously, adfreezing between the pile and the first (upper) frozen layer is restored fully or in part. The second, freshly frozen layer, as well as the first layer which has gained additional water, increases in volume and causes additional swelling of the ground. The surface level rises additionally to $+\Delta h_1$ and assumes position 4 (Fig. 23). The pile is subjected now to a new heaving force N_1. This force is greater than N because the swelling layer is thicker, since both the given layer h_1 and partly the first layer h are effective, and since freezing probably occurs now at a larger inflow of water. This force is resisted by the resultant force $\sum S_1 = T_1 + Q + P$ which is smaller than $\sum S$ because T_1 decreases as freezing proceeds in depth.

The new heaving force N_1 is transmitted to the pile through the corresponding surface of adfreezing between the pile and the ground. The magnitude of this surface is generally indeterminate and is approximately equal to the thickness h_1 of the layer freshly frozen during the given time interval plus part of or the entire thickness h of the first layer. As in the previous cycle, if the adfreezing between the pile and the ground is weak, the force N_1 cannot be transmitted to the pile and

the force ΣS_1 causes cleavage between the adfrozen ground and the pile and recurrence of the gap around the pile; the ground assumes position 5 (Fig. 23).

A similar process occurs during freezing of a third layer h_2 and results in position 6 (Fig. 23). The ground surface rises to position $+\Delta h_2$. A new heaving force N_2 arises. The magnitude of the adfreezing surface is determined now from the pile diameter and the height h_2 plus some indefinite part of the sum $h + h_1$. If the adfreezing strength still is insignificant and does not exceed $\Sigma S_2 = T_2 + Q + P$, no uplift of the pile will occur. However if the adfreezing between the pile and the frozen ground is considerable, and the resultant total strength of adfreezing exceeds the resisting force ΣS_2, the pile will pull out of the ground a distance of $\Delta h_2 + \Delta h_1$; that is, the pile will heave. Further freezing of successive ground layers will result in increased protrusion of the pile from the ground.

Water and fluid ground usually fill the void formed when the pile has been uplifted. The water freezes and forms ice beneath the pile (Fig. 24). This ice freezes in summer and the pile settles slightly. The fluid ground which flowed into the void remained there, so that the pile cannot resume its original position. The pile remains protruded, that is, it retains "residual heaving." Recurrence of this phenomenon over a period of several years causes the pile to be forced completely out of the ground. Figure 25 shows heaved fence posts. It is clearly seen that the heaving is not uniform.

In accordance with the preceding, it seems correct and logical to assume that the heaving force is

$$N = H \tau p$$

where $\underline{H}$ is the height of adfreezing between the pile and the active layer (Fig. 26), τ is the corresponding mean adfreezing strength, and $\underline{p}$ is the perimeter of the pile section. At definite values of $\underline{H}$ and τ (Fig. 26), determined from local factors, heaving of the pile would occur if the following condition is satisfied:

$$H \tau p \geq T + Q + P \qquad (1)$$

In other words, heaving would occur if the depth $\underline{H}$ of adfreezing between

the ground and the pile of given cross section as well as the adfreezing strength τ are sufficient to transmit a heaving force capable of overcoming the resistance to uplift, that is, $T + Q + P$. Tsytovich's method for determining uplift is based on these considerations. This method is logical and technically correct. However, even now it is difficult to determine the proper magnitudes of $\underline{H}$ and τ to be used in the calculations.

Original calculations of the safety factor assume that $\underline{H}$ is equal to the entire depth of the active layer, while τ is computed from laboratory results. The latter value depends on the ice content and temperature of the adfreezing ground. The degree of saturation with ice was approximately determined from the formula

$$J = \frac{W}{W_n}$$

in which $\underline{W}$ is the ice (water) content of the ground by weight (moisture of the frozen ground) and W_n is the water-holding capacity of this ground after thawing.

The OST Manual 90032-39 recommends that τ be determined in such cases from Table IV which appears on page 146 of this text. Use of such values of $\underline{H}$ and τ leads to rather parodoxical results. Even if relatively favorable conditions are assumed, such as an active layer of 200 cm in thickness, for example, and τ is assigned a value corresponding to unity saturation with ice and determined from the OST table, then the heaving force on a pile of diameter $d = 20$ cm at a ground temperature of only $-1°$ C will be

$$N = 3.14 \times 20 \times 200 \times 6 = 75360 \text{ kg}$$

At the same time, the tensile strength of such a timber pile (of poor quality) is

$$E = \frac{3.14 \times 20^2}{4} 200 = 62800 \text{ kg}$$

Since $N \geq E$, it need be assumed that heaving can result in tensile failure of the pile, which never happens in actuality. $\underline{N}$ might possibly be less than $\underline{E}$ in other cases, but even then it would amount to several tens of tons, whereas the load on a pile of diameter $d = 20$ cm seldom exceeds 10 tons, hence all piles should be subject to heaving, which is not usually the case. On the other hand, all attempts to design

piles that would resist heaving determined in accordance with the preceding method invariably result in design and layout that cannot be justified in spite of the perfect logic involved. Evidently, the values of H and τ used in the computation are erroneous. Apparently, they are greatly overestimated.

Field observations have repeatedly shown that the upper part of the active layer does not adfreeze to the pile when the frost penetrates deeper into the ground. N. I. Bikov, who devoted a great deal of time to the study of pile heaving, assumes that the process of swelling occurs only in 60 to 75 per cent of the active layer in regions where this layer is 2.0 to 2.5 m deep, in spite of the fact that the moisture content is 30 per cent. Having investigated the swelling process and the corresponding available data, K. D. Morozov [3, page 49] arrived at a similar conclusion.

With regards to the magnitude τ, recent studies prove that the OST data, mentioned previously, are inadequate for determining the strength of adfreezing between the pile and the ground under natural conditions. Bikov has conducted a series of experiments at a permafrost station and found that the adfreezing strength of the ground at this station did not exceed 120 to 200 kg per cm of pile perimeter; this result, coupled with the fact that the swelling layer is only 150 cm thick, yields a maximum adfreezing strength of 1.3 kg per sq cm in the case of the relatively poor ground involved, consisting of supersaturated silty loam. In general, his experimental results show that the adfreezing strength averages between 0.6 to 1.3 kg per sq cm.

According to V. K. Yanovsky, who conducted a series of tests along the Vorkuta River to determine the adfreezing between timber piles and ground, this strength ranges from 0.8 to 1.2 kg per sq cm. P. I. Melnikov conducted similar tests in Yakutia under natural conditions and found that the adfreezing strength was about 1 kg per sq cm. Additional tests must be conducted in order to determine the final value of τ for various types of ground under various conditions. In the meantime, it is expedient to use the values for adfreezing strength given in Table V; they are valid for preliminary evaluation, since they

are related to ice saturation and temperature of the ground, are based on the test results described above, and are derived in accordance with the relationship between ice saturation and τ, established in OST Manual 90032-39.

Table V

COMPUTATIONAL VALUES OF TANGENTIAL ADFREEZING STRENGTH τ IN KG PER SQ CM

Adfreezing Surfaces	Temperature -1° C Ice Saturation				Temperature -10° C Ice Saturation			
	0.25	0.50	0.75	1 to 1.4	0.25	0.50	0.75	1 to 1.4
Fine-textured ground (loamy sand, sandy loam, clay loam, silt) and wood	0.40	0.50	0.70	1.00	0.50	1.20	2.20	2.60
The same ground and concrete	0.40	0.60	0.80	1.20	0.60	1.40	2.60	3.20

It should be noted that most intensive swelling occurs not immediately after the onset of frost, but about a month later. Therefore, it is judicious arbitrarily to divide the assumed thickness of the frozen layer into two parts, one with lower and the other relatively higher temperature. Investigation of the mean temperatures at various depths of the active layer in various places shows that the temperature of the upper half of the frozen layer may be taken as -10° C, while that of the lower layer may be taken as -1° C. Accordingly, the following formula is recommended for determining the heaving force

$$R = \frac{0.75hp\tau}{2} + \frac{0.75hp\tau_1}{2} \approx 0.38hp(\tau + \tau_1) \qquad (2)$$

in which $\underline{h}$ is the thickness of the active layer in cm, $\underline{p}$ is the perimeter of the foundation in cm, τ is the adfreezing strength in kg per sq cm at a temperature of -1° C (from Table V), and τ_1 is the adfreezing strength in kg per sq cm at a temperature of -10° C (from Table V).

Application to the preceding example yields

$$R = 0.38 \times 200 \times 3.14 \times 20(1.20 + 3.20) = 21000 \text{ kg}$$

This magnitude of the heaving force seems to be a fair approximation of its actual value.

It need be emphasized that the data of Table V may be used only for preliminary estimates. These data should definitely not be used for purposes of final evaluation. The design value of the adfreezing strength must be determined in each instance by means of careful field tests.

2. Deformation of Structures

Posts are used in building construction either as columns sunk into the ground or as piles. Figure 27 shows a typical design of a frame dwelling supported on timber footings extended into the ground. Buildings of this type are frequently encountered in the permafrost regions, since they were adopted during initial construction of railroads. The drawing shows all the design aspects.

Heaving of the footings of such buildings results in the deformation shown schematically in Fig. 28. The effects of such heaving on a wooden building are as follows: the building tilts, the floors buckle, the timber frames part, and cracks are formed. During the warm season the heaved structure settles and tends to resume its previous position. As stated previously, the space beneath the footing is occasionally filled with ground, so that no settling or incomplete settling occurs. Heaving recurs during the following winter, and the same footings or other footings undergo deformation. The result is that the building soon becomes useless in spite of annual repairs and improvements.

Masonry piers are also subject to heaving. When such a pier extends deep in the ground, it forms a firm bond with the ground beneath the active layer because of friction or adfreezing to the permafrost; swelling of the upper portion of the active layer may cause rupture of the pier and occurrence of horizontal cracks (Fig. 29). The cracks may fill with fluid ground, in which case during the summer the structure would retain the position it assumed during winter heaving. If this does not occur, the structure would settle and assume its

original position, while the cracks would close. At any rate, it is obvious that annual recurrence of nonuniform heaving may rapidly render the structure entirely useless even in this case.

Timber piles of trestles, bridges, and similar structures, frequently undergo heaving during the winter if they are driven insufficiently deep into the ground. Deformation of pile structures causes failure of collars and tie beams, as well as complete deformation of most notches and mortises. As in the case of posts, heaving of piles to the ultimate height occurs intermittently during the entire freezing period of the active layer. Heaving of footings is most extensive when the active layer consists of fine-textured material. Heaving of piles often occurs even in the case of the sandy active layer.

The maximum recorded heaving of bridge piers and piles was about 2 m. During a single winter, however, the extent of heaving is only 40 to 60 cm. Heaved piles usually settle during the summer, but they almost never assume their original position; as a result, considerable residual deformation accumulates over a period of years, interfering with the normal functioning of the structure.

Pile systems do not heave uniformly in transverse and longitudinal directions. In the case of bridges, the piles located within the embankment do not heave if the fill is 1.5 to 2.5 m above ground level. Piles located in a stream rarely undergo deformation if the ice layer is about 1 m thick. Piles driven into the ground at an angle usually do not heave even when they are not deep. Bikov reports that piles driven into the permafrost to a depth of 2 m are not subject to heaving.

Piles driven into the talik beneath the active layer at places where permafrost is absent or occurs at great depth may be heaved in some cases (Fig. 30) if they do not extend sufficiently below the active layer and if this layer is extremely wet. Deformation does not occur if the piles are driven a sufficient distance, 3 to 4 m, below the active layer (a distance exceeding the thickness of the active layer by 1 to 1.5 m), that is, if the piles are driven to a depth of 5 to 6 m below

the surface. Deformation is particularly extensive and common in the case of structures supported on mats. Mats were widely used in bridge construction, for example, when permafrost or rock occurred directly beneath the active layer.

Most current structures founded on mats have deformed, in spite of the fact that the swelling ground around the piles was removed and the space was filled with gravel in order to reduce heaving (Fig. 31). Such systems are deformed because the posts are torn away from the mats; while the mats remain in place, the posts are heaved to a certain height (Fig. 32). The clamps joining the posts to the mat become straightened and pull out of the wood, while the bolts in the collar become twisted and pull out. If the joints are extremely strong, the mats themselves rise slightly; this is preceded by extensive deformation of the timber and the metal fastenings. Experience shows that mats for foundation columns may be used only under particularly favorable conditions, such as in the case when the mats are located in dry ground that drains well.

Heaving always causes rupture of pile splices located within the active layer, so that the upper part of the pile is heaved even if the ordinary fastenings are extremely strong. Deformation of this type is shown in Fig. 33. The splices of the extreme left and right bridge piles are ruptured; the displacement is about 60 cm. The collars are pulled clear and the bolts are withdrawn from the wood. Although the immediate ground has been removed, the piles did not settle back in place because of the extensive deformation of the upper system.

Figure 34 shows an exterior view of a deformed railway bridge supported on piles. Upon heaving, the bridge became humped; the hump is clearly seen in the photograph.

Also masonry structures frequently are affected by swelling of the active layer. Nonuniform heaving of the foundations causes cracks in the walls. However, such deformation is relatively rare and occurs in the case of unheated masonry structures. Most masonry buildings are

heated, so that swelling of the ground near the buildings, which can occur even under these conditions, is not so intense. This is explained by the fact that masonry buildings usually are erected in relatively populated areas in which meliorative drainage operations reduced the water content of the ground. In addition, the building itself is a source of heat which is partially transferred to the ground; this prevents rapid penetration of cold into the ground and generally increases the temperature of the ground near the foundations, so that strong adfreezing cannot occur. For this reason, heaving of foundations of masonry buildings occurs only in cases where conditions are unfavorable, particularly if ~~no~~ appropriate setps are taken to reduce the hazard of deformation.

An interesting case of deformation due to heaving of a masonry building occurred in Yakutsk in 1913. This building, which housed the technical high school, had brick walls. After two stories had been erected, the building was left uncompleted over the winter. Considerable heaving occurred during the winter. After repairs were made, construction was completed, and the building was occupied, deformation no longer occurred.

The following data about the deformation of masonry piers of a bridge are indicative of the magnitude of the heaving force. Figure 35 shows a stone pier of a railway bridge with spans of 10.7 m; the pier had been sunk to a depth of 2.4 m in the permafrost region. Heaving occurred annually. Within 15 years the pier was raised a distance of 128 cm. The ground around the pier was sandy loam. Heaving began regularly at the end of October, became more intense in January and February, and ceased in March or April. The pier hardly settled during the summer. Boreholes showed that fluid ground and water accumulated beneath the pier base; these materials froze and prevented settling of the pier. The surrounding ground was supersaturated. Permafrost at this point occurred at a depth of 1.6 to 1.7 m.

Figure 36 shows another interesting example of heaving of a bridge pier; the spans involved are 6.4 m in length. This pier was sunk deep into rock. Its base was 4.3 m below the surface. There was no seepage into the foundation pit and no pumping was required during

construction of the pier. The pier became deformed several years after it was built. Cracks were found in the masonry. In order to determine the condition of the structure, the ground adjoining the upper part of the foundation was removed; horizontal cracks were found in the upper third of the pier. No cracks were found in the lower part, although the ground was cleared away to a depth of 4 m. The horizontal cracks evidently resulted from ground swelling which heaved the upper part of the pier and separated it from the lower part. The lower part, which was heavy and wedged deep in the ground, heaved very little. During June the pier settled, but not completely.

Figure 37 shows a culvert deformed by ground swelling. The culvert rests on separate foundations. Disproportionate and intensive heaving of the individual foundations, resulting from excessive moisture in the ground, caused the arch to crack; the cracks later extended to the walls above the culvert.

Swelling ground usually deforms the roof and sloping wingwalls of the culvert. The wingwalls nearly always have vertical cracks which widen towards the foundation base; the cracks are particularly numerous at the joint between the first layer of bricks and the keystone. During swelling of the ground the wingwalls act as cantilevers and, since masonry is not strong in tension, they separate from the culvert at the junction.

Horizontal stresses developed in the process of heaving are known to have caused failure of stone foundations in some cases. However such forces can arise only under special conditions, so that deformation of this type is quite rare. V. A. Svinin [5] , the engineer who first discussed this aspect, reasons that in the case of a heated building the thawed ground within some closed periphery of the foundations cannot prevent expansion of the freezing ground outside. This results in an unbalanced horizontal pressure (Fig. 38) directed towards the building. This pressure causes the foundations to deflect inward and cracks are formed in them. Deformation of the foundation causes cracks to occur in the walls also. Figure 39 shows an uncovered foundation of such a building. Crushed stonework and broken masonry

are readily visible.

In an effort to adapt construction to changes in surface level, construction engineers made extensive use of crib foundations for wooden building (Fig. 40). These are cribs of short logs, arranged crisscross, on which the bottom row of timbers rests at several places. These cribs were laid directly on the ground from which the cover of moss and humus has been removed. Sometimes the ground beneath the cribs is excavated to some depth and is replaced by rubble, gravel, or sand. It is assumed that a building supported in this manner would rise and subside in conformity with the changes in ground level during swelling, and that its relative rigidity would tend to modify these changes.

However, recent experience has shown that most heated buildings were greatly deformed regardless of arrangement, type of cribs, and the nature of the ground beneath the cribs. The deformation affected the buildings as follows. Gaps formed between wall timbers, frame joints came apart, some buildings tilted excessively and warped, floor planks separated, and windows and doors became stuck. Some wooden buildings gradually began to sink into the ground. All these occurrences are due to the fact that the active layer often is exceedingly wet and has low bearing capacity. It swells extensively in the winter, and undergoes considerable settling under load in the summer. Therefore, simple crib foundations are unsuitable and are currently used only in the case of wooden buildings of secondary importance that are mostly unheated.

This consideration led to the design of buildings on crib foundations over a special fill (Fig. 41) of well-draining earth or slag covered with a layer of peat or moss. Such fill prevents rapid penetration of cold into the ground and tends to decrease the temperature fluctuations of the ground beneath the building and near it; in addition, it forms a layer which can distribute the weight of the building over a large area of the active layer, and tends to reduce the moisture of the ground near the building. Theoretically, a suitable fill should make it possible to maintain a constant temperature of about zero in the upper portion of the active layer; however, this would require a fairly thick fill, even if the fill material had good thermal

resistance. The effectiveness of such a design has not been established, since such buildings are infrequent. It is reported that a house of this type exists in Skovorodino. It has a ventilated air space over a crib foundation laid on a slag fill 70 cm thick. Nevertheless, this house has been subjected to heaving.

C. Deformation of Structures Due to Thawing of the Permafrost

1. General Remarks

It was stated previously that the upper permafrost limit is unstable even under natural conditions of the region involved, and that it is affected by relatively insignificant factors. It is obvious, therefore, that human activity would cause considerable changes in the depth at which permafrost occurs. Arrival of man is accompanied by economic activity which greatly disturbs the natural regime and character of the region. Man builds roads, buildings, clears the ground, and tills the soil. As a result, the moss and vegetation covers of the ground are destroyed, forests are logged, and the ground is drained near construction sites. Various structures are erected on the ground and foreign bodies, such as foundations and conduits, are introduced into the active layer and the permafrost. Road building and grading of sites selected for settlements result in cuts and fills. All those activities produce a rapid and extensive change in the initial regime of the ground water, in moisture content of the ground, and in the natural temperature regime of the ground, so that the upper permafrost limit is displaced with respect to its position prior to the development of the given region or area.

Such activities usually cause recession of the upper permafrost limit; in individual cases at particularly favorable conditions, however, the level at which the permafrost occurs rises and approaches the ground surface. The variation of the upper permafrost limit is rather indefinite and ranges within several meters. It is nearly impossible to predict the extent to which the permafrost level may drop since this phenomenon largely depends on both the regime of ground water and the surface cover, so that the factor of thermal exchange between the ground and the atmosphere is involved.

Some engineers are of the opinion that the change in the upper permafrost limit can be determined experimentally in places selected as construction sites. Of course, this is feasible in principle. However, the usual test method involved is erroneous and may produce misleading results. This method is as follows. An area of about 30 by 30 m at a given location is stripped of vegetation, and its moss and peat covers are removed; it is assumed that the level of the upper permafrost limit, established during one year under condition of such bare surface, would approximate the level which this limit would attain after completion of construction in the region.

In many cases a test area of such dimensions will not provide even an approximate idea of the possible changes in permafrost level. Simple removal of the surface cover from a small area confined within a limitless region of virgin ground cover yields incorrect and unexpected results which have nothing in common with actuality, since the regime of the ground water under such test conditions is altered in a manner totally different from that occurring when a large area is developed and is covered with structures. It should be noted that the results of such tests would probably vary in accordance with the season (spring or fall) in which the ground was bared and observations began. One year is adequate for completion of the cycle of the phenomena occurring in such an area. This method is applicable only in the case of isolated, small structures, provided a much larger test area is used.

It is judicious to solve the problem of the position of the upper permafrost limit on the basis of actual construction experience in the given or analogous regions, and to proceed with construction only after the entire area has been cleared and graded and all meliorative measures for subsequent use have been accomplished. These preparatory operations should be carried out at least one year prior to construction.

It should be noted that the data, obtained by O. I. Fink and others on construction of the Amur Railroad, indicate that thawing of the permafrost and, consequently, lowering of the upper permafrost limit in regions under development continue for a period of four to five years in the case of ordinary ground, and two to three years in the case of

sandy ground. According to Fink, the maximal depths of thawing along the Amur and Trans-Baikal Railroads, determined by various methods at various points, average as follows: 3.5 m, when determined by excavating, drilling or probing; 3.8 m when determined by thermometer readings. In places where the surface cover has been disturbed, the average value increased to 4.3 m and the maximum increased to 5.1 m. These values refer approximately to regions in which the average annual atmospheric temperature ranges from 1.8° to 2.5° C and the snow cover is 25 to 45 cm thick. These values are not valid for other regions. In shady places or on slopes facing north, these magnitudes are 15 to 20 per cent smaller; on sunny slopes, however, they are 10 to 15 per cent larger.

Table VI, containing data that have been verified reasonably well, shows the extent of change in the upper permafrost limit beneath buildings.

TABLE VI

SPECIFIC INSTANCES OF CHANGES IN POSITION OF UPPER PERMAFROST LIMIT BENEATH BUILDINGS

No.	Building Location and Function	Depth of Upper Permafrost Limit below Ground Surface, in Meters		Remarks
		During Construction	After Construction	
1	Heated depot at Skovorodino Station:	4	8	12 years later
2	Residence	2.5	2	6 years later
3	Carhouse	h	h -0.74	Rise in level
4	Unheated building	h	h -1.10	Rise in level
5	Power plant	1.2	6.7	2 years later
6	Experimental frame residence on timber piles	2.5	1.6	1 year later
7	Reservoir at Magadachi at Petrovsk-Zabaikalsk:	h	h+3	2 years later
8	Experimental masonry residence on rubble foundations	2.5	3.20	3 years later
9	Heated depot	2.5	10.0	
10	Unheated depot	2.5	2.5	20 years later
11	Passenger station	2.5	3.0	Deformed in 8 years
12	Passenger station	2.5	-	Collapsed

2. Deformation of Structures

Deformation of structures erected on permafrost is due primarily to the fact that the frozen layer thaws, its upper limit is lowered and, consequently, building foundations become based on thawed ground. The latter is quite weak and wet (often comprising silt and slud) so it cannot carry the foundation load; therefore, it settles, and cracks occur in the foundations and walls.

Decreasing the unit load on the foundation area does not prevent deformation, since the frozen ground usually has a high moisture content and thawing is accompanied by considerable decrease in volume so that the ground would settle even if it were not subjected to the weight of the building. The uppermost layer of the permafrost is the wettest, and it is also the layer on which foundations usually rest. A structure may settle even if the permafrost does not thaw; this occurs when the permafrost comprises fine-textured and supersaturated material, and if its temperature is near zero (up to -0.5° C).

Sandy ground, and particularly gravel and pebbles, settle relatively little upon thawing even when they are supersaturated; when their moisture content is less than 30 per cent by weight, settling is hardly noticeable and does not constitute a hazard to structures, provided the ground contains no ice lenses. Melting of ice lenses or of large quantities of imbedded ice causes collapse of the ground; the extent of such caving increases with increasing ice content.

Thermokarst depressions resulting from melting of imbedded ice are very large. Figure 42 shows such a depression in a temporary road built on a continuous log mat. This deformation occurred at the end of summer, when a railway cut was made near the temporary road, which created conditions facilitating the thawing of the permafrost and the ice lenses near the cut.

O. I. Fink reports a unique case of considerable settling of the ground due to thawing of the frozen layer and melting of the ground ice, caused by an apparently insignificant circumstance. The event occurred during construction of a railroad. Near a siding, cattle

had trampled a trail on a swampy mar and had partially destroyed the surface cover near the trail. During the summer the surface along the trail settled more than 70 cm. This depression constituted a new runoff channel, so that a nearby bridge spanned a dry bed. Moreover, the road embankment was washed away later during torrential rains. Thus, a relatively insignificant factor caused serious complications.

In another instance, drainage canals were dug to dry an area allocated for industrial development. This was done without taking into consideration the moisture content and composition of the ground. Since the ground was silty and slud-like, and it contained large quantities of ice, intense thawing and solifluction occurred. The canals became huge washouts 2.5 m deep and 4.5 m wide, in which a mass of fluid ground was moving.

In addition to the general effect on the upper permafrost limit due to erection of a series of structures in a given locality, each building has a separate effect on the level of this limit near the building or beneath it, depending on the particular design and function of the building. In the case of buildings erected on a continuous rubble foundation, the temperature of the ground beneath the entire construction area usually rises and the upper permafrost limit is correspondingly lowered. N. A. Tsytovich established this fact on the basis of data accumulated during three years of observation of ground temperature beneath the experimental building at the permafrost station of Petrovsk-Zabaialsk. This building had continuous rubble foundations extending to a depth of 2.75 m and penetrating the permafrost layer to a depth of 0.5 m. The house had brick walls and wooden floors and ceiling. The frozen ground beneath this building thawed to a depth of about 1 m at the south side, so that part of the foundation rested on thawed ground, as shown in Fig. 43.

Heat is conducted to the ground partly through the building foundations. The walls are heated by the sun in the summer and by stoves and other heat sources in the winter. The heat from the walls is transferred to the foundations and causes thawing of the frozen ground at the base. In addition, the entire mass of the building

absorbs and radiates heat which is partly transferred to the ground. M. I. Sumgin discovered that the thermal effect of the rubble foundations of the aforementioned experimental building on the permafrost beneath the building was negligible in comparison with the effect of the entire area beneath the structure.

It is doubtful whether this is entirely true, since another experimental building, made of wood and erected in another region on timber piles (Fig. 44) extending 2.5 m into the permafrost layer, had no negative effect on the level of the upper permafrost limit. On the contrary, a rise in level occurred in this case. It is true that the second house was a frame structure and its air space was open in the winter, while the first house was a masonry structure and its air space had to be ventilated in the summer [2, page 349].

Comparison of these two buildings is difficult because of difference in material and design. It is probable that the permafrost beneath the masonry building would not have thawed, despite the material and foundation design involved, if it had a ventilated air space. On the other hand, the permafrost beneath the frame building might have thawed if this building had no ventilated air space.

It is known that the ground beneath buildings is warmed and the upper permafrost limit is lowered even when the building has no regular foundations, such as in the case of crib foundations. Nevertheless, the thermal effect of foundations on the permafrost should not be neglected. This effect is not negligible, as proved by the fact that permafrost thaws beneath masonry foundations of unheated structures and, in particular, beneath bridge piers.

Thawing of permafrost beneath buildings is usually most extensive at the south side; a rise in permafrost level may occur at the north side. This is due to the fact that the south side of a structure is readily heated by the sun and the ground nearby is exposed to the sun. On the north side, however, the ground is shaded by the building and the sun does not heat the structure as readily.

Figure 45 shows a typical change in the upper permafrost limit. It is based on data obtained by I. D. Belokrylov and M. I. Sumgin.

The latter notes that formation of thawed ground, in the shape of a bowl or trough, beneath heated dwellings is a phenomenon common to permafrost regions south of the 55th parallel. As a result of such thawing, the routh wall of a building settles and cracks appear in it; the building itself tilts in the direction of the settling.

The best known case of deformation is that which occurred at a certain railroad shop where settling ranged from 10 to 30 cm and the walls had numerous cracks and were tilted. South walls and corners were affected most. It should be noted that thawing of the permafrost and settling of the foundations occurred, in this case, even before the buildings were heated.

Many heated buildings begin to settle not immediately after construction, but several years afterward. This is due to the following two factors: (1) warming of the ground is gradual and the effect reaches the foundation base only after a relatively large time interval; (2) the particular design of the building facilitates redistribution of the pressure on the ground and resists the stresses occurring during settling. Thus, for example, a passenger depot of masonry construction on continuous foundations began to deform seven years after it was built. This structure was erected on a sand and gravel base 2.50 m wide and 0.85 m thick. A concrete slab 0.65 m thick and reinforced by steel rails was placed under the entire foundation. No measures were taken to prevent heat transfer into the ground. Moreover, cesspools and a storage cellar were dug beneath the building. Cracks appeared in the middle section of the south wall seven years after completion of construction. No cracks could be detected in the remaining walls even thirteen years after construction.

A certain roundhouse has not been heated for five years after construction and showed no deformation. One year after heating installation, cracks occurred in the walls and part of the walls had to be pulled down. The same happened at other depots where use of hot water for flushing locomotives and installation of heating resulted in deformation of the structures.

Even unheated masonry structures often deform. The old, unheated depot shown in Fig. 46, is a representative example; the south side of the building underwent catastrophic settling. The nature and extent of the strains are readily seen. The deformation was due to a change in the natural temperature regime of the ground.

The intensity and extent of thawing of the frozen layer increase considerably if cesspools or warm cellars are excavated beneath the building, and if water pipes, sewage pipes, or heating mains pass through the foundations. According to Professor M. I. Evdokimov-Rokotovsky, a building housing a small power station became deformed because a shallow well was dug there for the purpose of cooling the hot water from the Diesel engines.

Numerous difficulties arise from the fact that waste water of industrial plants is not piped away from the buildings but is discharged on the ground in the immediate vicinity of the buildings. This practice always causes lowering of the permafrost level and catastrophic settling of the foundations. The drop in permafrost level is particularly large in the part of the building where the heat sources are located. Thus, the walls of the steam boiler room in a locomotive roundhouse settled 80 cm. In one fish cannery, the ground beneath the furnace room thawed so rapidly that the floor settled more than 50 cm and the walls were greatly deformed.

Deformation of bridge piers and buttresses is a fairly common occurrence; it is due to settling resulting from lowering of the upper permafrost limit. Piers and abutments of double track bridges undergo particularly large deformation. Such deformation occurs in the form of cracks, tilting, and even sidewise displacement of the entire structure. Observations show that the largest deformation occurs on the south side of the supports. Thawing of the ground beneath piers causes settling and tilting of the piers. Vertical cracks, widening towards the top, occur in the masonry. Figure 47 shows the deformation of an abutment of a double track bridge with a span of 17 m. The deformation is more pronounced and the cracks are more numerous and larger on the left side. Apparently, thawing of the permafrost is more extensive there.

Figure 48 shows the probable change in the upper permafrost limit. The depth of thawing must have been greater in front of the abutment, while the upper permafrost limit probably extended upward into the fill at the rear of the abutment. Under these conditions, the abutment would not only form cracks, but would also tend to slide toward the stream and squeeze the bridge girders between the encasing walls. Laterally, thawing is more extensive on the south side than on the north side and is least in the middle. In the transverse section in front of the abutment, the highest point of the upper permafrost limit occurs to the right of the axis of symmetry. Accordingly, the permafrost surface beneath the pier constitutes a complex curved surface which slopes generally in the direction of one of the corners of the abutment, while sloping gently in the direction of the opposite corner; as a result, the abutment is likely to tilt laterally and to settle, so that large cracks would occur in the masonry on the south side and smaller cracks on the north side.

It is equally probable that the abutment would shift in the direction where the thawing is most extensive. The horizontal pressure of the fill, the weight of the train on the rupture prism, and the dynamic effect of the train load would intensify the displacement of the entire abutment forward and laterally. As stated previously, deformation is particularly extensive in the case of wide piers of double track bridges. This is due to the fact that a pier of larger lateral dimension acts as a double-cantilever beam and cannot withstand the relatively large stresses. Under such conditions, therefore, it is advisable to use separate abutments for each track.

D. Deformation of Structures Due to Icing Processes

Ground icings constitute the greatest hazard to buildings. The icing near a building may form a mound that would tilt the building, bend it, or damage some portion of it. Figure 49 shows an icing near a building at a railway station, which caused extensive damage. Formation of icing mounds in areas occupied by structures is due to high moisture content of the ground and the presence of ground water.

The places where such icing might form are indeterminate and cannot be predicted with precision. Icing mounds and swellings of the ground occur in different places at different times. Near Urusha, for example, mounds formed in three different places during the brief period between 1926 and 1930, and they reached a height of 2 m within three to four days. One of these mounds caused slight deformation of a warehouse.

A careful investigation of a given locality usually can determine whether ground icings are likely to occur in this area. On the other hand, it should be noted that erection of structures may create such conditions that icings and icing mounds would form in places where they had not formed previously. This is due to the fact that buildings or other structures cause changes in the hydrological condition of the locality and the temperature regime of the ground. To avoid complications, it is evidently necessary to pay particular attention to proper selection of construction sites; areas where icing processes occur should be avoided, and wherever icings do not occur it is judicious to take measures to prevent the occurrence of icings after structures have been erected in the area.

There are cases when icing processes are manifested in other ways. When frost causes the active layer to freeze, the flow of ground water is compressed between the freezing upper ground layer and the underlying permafrost or another impervious layer. The water, which is under increased pressure due to this compression, tends to flow into places that have not yet been frozen. In populated regions, such places are the areas beneath buildings, since the buildings themselves and the heat which they radiate protect these areas against freezing. The water accumulates beneath the building and then bursts through the ground surface into the space beneath the floor; a portion of this water freezes there, while the remainder flows out. Since the water is under pressure, it breaks through the thin ice crust on the ground and continues to flow into the building. The occupants are compelled to abandon the building, the ice gradually fills the entire house (Fig. 50), and then forces its way through windows and doors. The flow of water and its conversion into ice are so rapid that there is not sufficient time to remove all

the furniture. The ice filling the house wrecks floors and ceilings. Figure 51 shows a house filled with icing ice. Having filled the entire structure, the water flowed out through apertures in the walls and formed icefalls which extended to the surface of the ground nearby. Removal of the building framework would expose a huge mold of ice (Fig. 52). Such a phenomenon usually occurs in heated buildings, since talik is formed within the permafrost beneath the building due to the building itself and its heat which penetrates into the ground; this facilitates the emergence of the icing water to the ground surface. Similar icing processes occur also in unheated buildings under special conditions and, particularly, when the ground water is under high pressure. However, this phenomenon is not common in the case of unheated buildings. Usually the spaces beneath the floors of such buildings are filled with water that seeps in from the adjacent ground layers. This may occur in either timber or masonry buildings.

The icing at Skovorodino is a rather unique instance of an icing resulting from erection of structures. This large icing, which flooded and deformed a number of buildings, formed in 1934 as a result of seepage of pond water through a dam and along its thawed base [6]. The icing began to form on February 4. After six days it had formed an ice field 1 m thick and covered an area of tens of thousands of square meters. Figure 53 shows a frame building which was nearly completely flooded by this icing. A sudden rise in temperature stopped the growth of this icing.

Deformation of railway and highway structures due to icings is more common than that of buildings. V. C. Petrov [7] has made a detailed study of the effect of icings on road structures. However, only the most essential information will be presented here.

Bridges and culverts comprise the railway structures which are usually damaged by icings. Icings are not common in the southern regions of permafrost where most railway lines are located. But as railroad construction is undertaken in the northern regions, many complications might arise due to icings if no advance provisions are made and corresponding measures taken to prevent their destructive effects.

In the southern region of permafrost, spring or river icings fill culverts under embankments and clog the openings underneath small bridges. The resulting deformation comprises breakage of and damage to protruding parts of struts, girders, etc. Explosion of a river icing mound can cause considerable damage to a bridge. The water emerging from an exploded icing mound usually carries slabs of ice weighing tens of tons and can readily destroy any structure in its path. Explosion of icing mounds in a given region occurs approximately at the same time each year. Most explosions along the roads in Yakutia occur at the end of February or the beginning of March.

According to V. G. Petrov, an icing mound at the Onon River exploded and scattered slabs of ice weighing nearly 205 tons each. Figure 6 shows a relatively small slab of ice from this mound, weighing 38 tons. Icings on roads in this area are quite common. There were 122 icings, of which 44 were very large, formed along a stretch of 725 km. They comprised 55 per cent river icings and 45 per cent ground icings, although many were of joint origin. The occurrence of such a large number of icings near the road was due to the roadbed itself. Construction of the road created conditions facilitating deeper freezing of the ground near the road, which blocked the flow of ground water.

River icings have a destructive effect upon bridges and often put them completely out of commission. In many cases icings overflow the roads, slowing traffic and making it hazardous. Traffic in such places is quite hazardous even in broad daylight because of the large areas covered by the icing water, the high icing mounds with deep crevasses up to 50 cm in width, and the constantly changing surface level of the icing (changes occur every few minutes). Traffic ceases completely at night, since there is danger of falling into a crevasse, immersion in the water at an air temperature of -30° C to -40° C, or negotiating a steep ice slope. Therefore, detours are necessary.

The following brief description of icings along an ordinary road, observed by V. G. Petrov, illustrates the extent of river icings. One of the largest icings along the road covered an area of 36,250 sq m and extended 275 m along the road. The ice reached a thickness of 2 m.

There were six mounds, of which the largest was 4 m high and 65 m in diameter. Permafrost occurred at a depth of 60 cm along the river bed and at a depth of 150 cm along the valley side. Another river icing covered an area of 36,875 sq m and extended 255 m along the road. The thickness of the ice ranged from 1 to 2 m. There were two mounds, of which the larger was 4 m high, 30 m wide, and 60 m long. Permafrost occurred at a depth of 80 cm.

Figure 54 shows an icing formed near a bridge along a road. This icing, of spring origin, reached the roadbed and spread along it a distance 216 m, and formed a mound near the bridge. A longitudinal fissure traversed the summit of the mound. The icing covered an area of 12,800 sq m. The thickness of the ice ranged from 0.5 to 1.0 m. The mound was 115 m long, 65 m wide, and 4 m high. The bridge was deformed, and the girders and cross-beams were displaced and partially broken.

Several small bridges along this road were completely engulfed and covered by the icing ice. In other cases, the crib piers were raised 50 to 60 cm and the bridge spans were shifted and deformed. Explosion of an icing mound may result in complete destruction of a bridge, as the crib piers are shifted and wrecked while the piles become sheared at ground level. Measures against icings are complicated because numerous river and spring icings continue to grow all winter without interruption even for a single day. Artificial removal of the ice is inadequate because its growth is very rapid. In surveying road routes, therefore, it is advisable to avoid places where icings are prevalent.

It should be noted, however, that after construction of a road or other structures has been completed, icings may form in places where they had not existed previously. This happened on the Amur-Yakutsk Road where, according to Petrov, 80 per cent of the icings resulted from the road building. Construction of bridges, culverts, fills, and cuts may readily create conditions facilitating the occurrence of icings, since such construction causes either freezing or a change in the flow direction of the ground water.

Underground structures, such as tunnels, require particular attention because ice tends to form in the tunnels due to seepage of water under pressure into seams and fissures of the facing. Figure 55 shows a tunnel in which ice formed early in the winter. Practically no water was encountered during excavation operations, but when the ground began to freeze in the fall, pressure caused the water to seep through the facing and to form stalactites and stalagmites upon freezing. The facing was partially damaged. Proper insulation of the facing and provisions for adequate drainage behind the walls along the entire tunnel could have prevented the damage. In some cases tunnel icings made it necessary to build gates at the tunnel entrances and to heat the tunnels during the winter. This arrangement facilitated removal of the water inside the tunnel by means of troughs extending along the tunnel. To avoid formation of icings at the entrances where the troughs discharged the water, it was necessary either to heat the troughs or to pump and collect the water in special wells, or even to heat the water.

E. Deformation of Earth Structures

1. General Considerations

The basic types of earth structures are fills and cuts in roads or in various other structures. Deformation of fills usually occurs in the form of extensive settling, creep, and sliding of part of the material forming the fill. Small fills are subject to swelling. In addition, fills are deformed by icings. Cuts are subject to caving and settling, while their walls often slide and creep and the bed becomes relatively fluid. Icings frequently disturb the normal state of cuts. However, swelling is the most common phenomenon and, consequently, constitutes the most important factor in the case of roads. All these forms of deformation are due primarily to the fact that construction operations have disturbed the natural temperature regime of the ground. The magnitude of the deformation depends on the moisture content and the composition of the ground in the earth structure or beneath it.

2. Deformation of Fills

The major cause of settling of fills overlying the permafrost is the thawing of the permafrost, that is the lowering of the upper permafrost limit beneath the fill and in its immediate vicinity. It seems reasonable to presume that if in a given area the permafrost occurs at a certain depth, deposition of a new layer on the ground surface would not only facilitate conservation of the permafrost, but would also tend to raise the upper permafrost limit. In actuality this is definitely not the case. Fills often cause lowering of the upper permafrost limit; the reason is as follows. Fills are usually made in the summer or, which is still worse, in the fall on thawed ground warmed by the air and sun. The fill material, which has a relatively high temperature and is deposited in relatively thin layers, is further heated by the sun. As a result, the temperature of the fill is much higher than the natural temperature of the ground underneath. Such a warm mass of material not only prevents the underlying natural ground from cooling at night, which occurs in most permafrost regions due to the considerable difference between daytime and nighttime temperatures, but continues to transfer its heat to the underlying ground during the night. Fills exceeding 5 m in height usually do not freeze during the first year and contain a thawed core with a relatively high temperature.

O. I. Fink cites the example of an earth dam in which the temperature during the first year, before water was admitted into the reservoir, was $+10^{\circ}$ C during January and February. At the top the dam froze to a depth of 4 m, while at the sides it froze to a depth of 3 to 3.5 m. By springtime the temperature in the core dropped to $+5^{\circ}$ C, but even at the beginning of June the temperature of the lower layers still was higher than that of the upper layers.

The heat accumulated in the bulk of the high fill during the first year after construction evidently facilitates thawing of the permafrost and settling of the ground under the weight of the fill. Subsequently, after the fill has deformed and the warm core has cooled, it is quite possible that the upper permafrost limit may resume its

original position and may even rise into the fill itself.

As stated previously, thawed permafrost usually has very low bearing capacity; hence, settling of a fill is a rational phenomenon. The magnitude of such settling comprises the following three components: (1) ordinary settling of the fill material, (2) decrease in volume due to conversion of ice into water, and (3) decrease in volume of the thawed layer due to compaction of the ground under the superimposed weight, accompanied by loss of ground water through squeezing process.

Ordinary settling of loose fill material is small, relatively uniform, and is compensated for by a corresponding increase in the original volume of the fill. The decrease in water volume due to conversion of ice into water is of considerable importance because permafrost is supersaturated and contains 25 to 50 per cent water by weight (the average is 30 to 40 per cent). Accordingly, the average moisture content is 400 to 500 liters per cu m of ground, so that conversion of the ice into water may decrease the volume of the ground by 4 to 5 per cent with respect to the thickness of the thawed ground. The decrease in volume of the ground due to compaction of the thawed layer and the loss of water may amount to 27 per cent of the thickness of the thawed layer, and may average 12 to 15 per cent.

Accordingly, total settling may reach 30 to 32 per cent of the thickness of the thawed layer, that is, the settling may range from 30 to 60 cm when the thawed permafrost layer is only 1 to 2 m thick. Such extensive and, moreover, nonuniform settling tends to disturb the equilibrium of the fill. Nonuniformity of settling is due to nonuniform thawing of the permafrost and heterogeneity of the ground with respect to composition, quality, and moisture content. The south side of the fill usually settles more than the north side because thawing of the permafrost is more intense at the south side.

Figure 56 shows the probable change in the upper permafrost limit beneath a high fill. The fact that the surface of the permafrost tends to slope constitutes a hazard to a structure because, in addition to settling, this may cause sliding of the fill along the sloping, wet, and slippery surface of the permafrost.

Melting of imbedded ice in the ground beneath the fill constitutes another cause of large settling accompanied by creep and sliding. This is most likely to occur in the case of fills up to 2 m in height, but it may occur also in the case of high fills. Melting of this ice is due primarily to the disturbed natural regime of the locality. Melting of ice lenses in areas under construction is often due to removal of moss and peat covers, establishment of supply dumps, and excavation of drainage ditches. Melting subsequently becomes more extensive, since the heat of the water from the thawed ground and the melted ice lenses affects other lenses and portions of the ice layer exposed at a given place.

A fill overlying a fairly deep ice lens will not deform if the following conditions prevail: if the fill is made in such a way that it does not constitute a large source of heat, if the surface cover is not destroyed and supply dumps are not established near the lens or the ice layer, and if the drainage ditches are located not less than 20 m from the foot of the fill. In addition, in order to avoid accumulation of water, it is necessary to grade the natural surface in a proper way and to build berms. Figure 42 illustrates the extent of possible settling.

Engineer S. V. Kovanko reports an instance of a washout 2 m deep and 5 to 10 m wide that occurred after spring rainfall on a site intended for a road fill. This washout was due to a ditch which discharged water from a hollow formed by removal of the moss cover. Inspection of the place revealed that frozen quicksand containing wedges of pure ice occurred at a depth of about 1 m beneath the peat layer. In spite of the fact that subsequently the washout was lined with peat and special clay dikes were built, a new and larger washout occurred during the summer.

The following fact demonstrates how frequently imbedded ice is encountered during construction of earth structures. Engineer E. I. Sukhodolsky reports that ice was found in 28 out of a total of 270 test holes made on a road section 45 km long. He is of the opinion that these figures do not represent the actual occurrence of

imbedded ice because the test holes were not uniformly distributed along the entire road, but were concentrated primarily near cuts, depot sites, and bridge crossings.

In some cases, fills are subjected to creep and sliding because the permafrost beneath the fill thaws less than at both ends (Fig. 57). Such thawing is due to destruction of surface cover at both sides of the fill as well as the existence of drainage ditches and material dumps. These factors facilitate deeper and more rapid thawing of the permafrost outside the fill than directly beneath the fill, particularly if the fill has been built during the spring. Small fills on the western section of the Amur Road, where the ground was saturated with ice, settled as much as 75 cm due to these factors.

According to V. V. Elenevsky, data obtained during road construction in the Far East prove that in some cases of high fills the upper permafrost limit may rise to such an extent that it penetrates into the fill. This has been confirmed to some extent by Afanasiev. The Turgutui fill verifies the fact that seasonal freezing can form within the body of the fill. Figure 58 illustrates the manner (in the form of a hump) in which the permafrost rises within the fill.

Formation of frost mounds within fills is possible if the fills are 2.5 to 3.0 m high and consist of poorly draining material. It should be noted that the rise in permafrost level may occur not immediately after construction of the fill, but several years later. The frost mound is usually displaced northward because the south side of the fill is warmed by the sun more than the north side. If the ground at the upper permafrost limit is supersaturated, the sloping surfaces of the frost mound within the fill may cause sliding of the upper thawed portion of the fill.

Railway embankments tend to slide when they are 5 m high or more. If the fill is lower, the frost mound will hardly rise more than 1 to 2 m, so that the resistance of the berm and the thawed portion of the slope is sufficient to prevent deformation. For this reason, small embankments deform rarely and only under conditions characterizing the deformation shown in Fig. 57.

In the case of embankments built entirely of well-draining material or containing such material at the base, the moisture content is low, inflow of water from beneath does not occur, and deformation is practically improbable even when the rise in permafrost level is considerable.

According to G. A. Nizovkin, a mass of frozen ground formed above and around a stone culvert in a fill 32 m high. He found a similar mass of frozen ground above a stone culvert in a fill on the Tomsk Road. Such a phenomenon, of course, may also result in deformation of the type described previously.

Deformation of fills often occurs due to seepage of water from beneath (Fig. 59). This occurs when the fills are built on mari or in places where wetting is likely. Water rises into the base of the fill by capillary action; as the base becomes wet, it cannot support the superimposed weight and it spreads. Accordingly, when the active layer freezes in the fall, the water occasionally tends to flow toward the base of the fill where the ground has not yet frozen; having saturated the bottom of the fill, the water flows out on the slopes and forms small icings. These icings constitute no direct hazard, except that they supersaturate the lower portion of the fill in the spring, which may result in deformation of the fill. Such deformation does not occur if the lower portion of the fill consists of well-draining material.

Swelling is a special type of fill deformation which, although small, is quite annoying and harmful. Swelling usually occurs in small fills (less than 2 m high) of earth material and is most extensive on mari and in tundra, where it interferes with normal functioning of the road. Sukhodolsky made a study of low road fills and found that, in addition to the known causes of swelling, seasonal migration of water within the fill occurs during annual freezing of fills composed of earth material. This water tends to move toward the source of the cold and usually accumulates near the layer underlying the ballast, so that further freezing of the body of the fill results in considerable deformation due to swelling.

In areas where icings tend to form, low fills are flooded with freezing water from the icings. It is difficult to remove the ice forming on the roadbed because new layers of ice form so rapidly that removal is not feasible. Icings render highways impassable during the winter and necessitate detours over the frozen earth. Maintenance of traffic on railroads necessitates persistent and hard labor of daily removal of the ice."

Figure 60 shows a ground icing covering a highway in Yakutia. Comparison between the height of the telegraph pole and that of the person standing near it shows that the pole is imbedded in ice to a depth equal to about half its height, or 3 to 4 m. The icing extends a distance of 390 m along the road, and a horse-and-wagon requires an hour to cover this distance [7] . It should be noted that deformation of a fill made of well-draining material is least frequent. In contrast, fills made of fine-textured material are subject to frequent and intense deformation. The stability of fills depends not only on the fill material, but also on the nature and particularly moisture content of the base, that is, the ground on which the fill is constructed. Earth structures located on mari, where the ground is extremely wet, are subject to most intense deformation.

The preceding established the fact that deformation of fills depends on their height and the material involved. Fills exceeding 5 m in height are more likely to deform than fills 2 to 5 m in height. Fills made of rock and coarse material drain well, so that they do not become saturated with water, do not swell, do not slide, and do not disintegrate during settling.

Earth dams constructed in permafrost regions deserve particular attention. Experience gained in construction of earth dams for water supply or other purposes revealed the following: (1) construction of such dams is extremely difficult because of the regime of the rivers involved; (2) it is extremely difficult to obtain impervious and stable foundations under permafrost conditions; (3) a large reservoir is not reliable as an artificial source of water supply because the high heat content of the large volume of water (this content is

high even at a relatively low temperature due to the extremely high specific heat) as well as the heat accumulated within the dam during construction cause deep thawing of the surrounding permafrost and of the foundation beneath the dam, which results in deformation and failure of the dam itself. Even in the case of bedrock, it is extremely difficult to render the foundation impermeable by forcing mortar into the cracks because the temperature of the ground is too low to facilitate hardening and setting of the mortar.

The permafrost beneath a reservoir at a railroad depot receded 3 m in the course of two years, while seepage through the bed was so intensive that the entire volume of water might have been lost if extraordinary measures had not been taken. A large drop in permafrost level occurred in another region within six months after construction of the dam. Extensive seepage through the thawed layer occurred despite the installation of a barrier of fine-textured material.

3. Deformation of Cuts

Deformation of cuts comprises primarily creep and sliding of the walls. Such deformation may continue for years. The largest deformation usually occurs in cuts made in the permafrost layer (Fig. 61). In this case, the upper permafrost limit recedes downward and assumes approximately the position shown in the diagram. Even if the permafrost does not consist of loam and silt, it usually is supersaturated, and its angle of repose usually is very small. Therefore, the sidewalls of the cut, located on a sloping and extremely wet permafrost surface, tend to slide toward the bottom of the cut. If the cut is made through loamy or silty ground, the resulting slides are still larger because the bottom of the cut also thaws, becomes fluid and completely loses its capacity to support the sidewalls. Figure 62 shows the sliding sidewalls of a railroad cut in permafrost.

The classic example of sidewall sliding in a railroad cut is the case in which deformation lasted forty-five years and was stopped only recently after installation of slag cushions designed by the engineer V. V. Elenevsky. The following description of this

deformation and of the factors involved was compiled by E. V. Pustovalov. The cut was made in a sandy loam and loamy sand detritus of diluvial and alluvial origin. It became clear during construction that the cut was passing through permafrost which would become completely fluid when thawed. After the permafrost had thawed, the bottom of the cut became a quagmire in which rails, ties, and even locomotives were sunk. It was very difficult to keep this cut open to traffic ever since its construction in 1900. Both sidewalls would slide at the beginning of every summer. The slope facing south caused more trouble during the first years than the opposite slope. Gradually its sliding diminished and ceased entirely in 1933. Sliding of the opposite sidewall continued until 1939. The following figures are indicative of the magnitude of the deformation involved. During the period from 1922 to 1932, the volume of material carted away from the cut amounted to 22,850 cu m. while the material that moved from the southern slope during the summer of 1938 was 5500 to 6000 cu m.

Various measures were taken to prevent sliding. In order to prevent the tracks from sinking into the relatively fluid ground at the bottom of the cut, the rails were laid on a cribwork of ties. Each summer the sidewalls of the cut were trimmed and their slope was made shallower, while deep stone drains were installed in them. Deep concrete and stone ditches were installed along the shoulders of the roadbed. These expensive measures to stabilize the sidewalls of the cut had no noticeable effect. The stone drains slid together with the sidewalls; at present there is nothing left of them but deep washouts in the sidewalls of the cut. The stone and concrete ditches were crushed and deformed. In spite of everything, the slides recurred annually and frequently caused interruption in traffic. Sliding usually occurred at the end of May or the beginning of June, when thawing reached a depth of 0.9 to 1.2 m. This coincided with the time of the first spring rains which further facilitated sliding. The fact that 1,200,000 rubles were allocated to improve the conditions in the cut is indicative of the difficulty and expense involved in controlling such deformation in the case of unsuitable ground.

In the case of fine-textured ground, the sidewalls of cuts often continue to slide even at slopes of 1:3 or 1:4. Imbedded ice at shallow depth in the bottom or sidewalls of a cut also causes creep, cavings, slides, and fluidity of the ground at the bottom.

Figure 63 shows a railroad cut filled with mud as a result of melting of imbedded ice. The ice in this cut occurred 2 m beneath the surface. It began to melt immediately after excavation started, and considerable sliding of the sidewalls occurred. Water accumulated in the cut. Ground was silty and supersaturated.

During construction of a certain section of this road, a cut was made in ground containing much ice. The result was that the bottom of the cut became quite fluid and the walls began to slide. According to engineer S. V. Kovanko, the trains could not pass through this cut even after seven layers of ties had been laid and sunk because the locomotive tended to sink somewhat and pushed a wave of fluid ground ahead of itself, which raised the track. Thus, the locomotive always moved uphill and depressed the track beneath it. Pumping and drainage installations were not effective. It was necessary to reroute the line. The line was laid on the downhill bank of the cut. This proved satisfactory and the tracks were left there, while the muddy and impassable cut was abandoned. Today this cut functions as a large ditch.

Considerable caving at the bottom of cuts occurred near the Tynda River.

The preceding proves that deformation of cuts is due primarily to high moisture content of the permafrost involved, the occurrence of imbedded ice, and the fact that the ground consists of fine-textured material which changes into slud when thawed. Ordinarily the slud has no bearing capacity, and both people and vehicles sink in it. In addition, ground of this type has a small angle of repose.

Occasionally frozen clay loam or sandy loam thaws under the dynamic effect of motion of people and vehicles, becomes plastic, tacky, and sticky. Traffic becomes impossible in such a case because motion causes the entire bottom of the cut to oscillate.

With regard to swelling in cuts, it is obvious that it can readily occur under such conditions. The types of ground mentioned above are supersaturated and swell readily. Swelling in cuts is extensive and it is feasible but difficult to control it. Swelling has been avoided in many cases by removing a layer 2 to 3 m thick from the bottom of the cut, replacing it with well-draining material, and installing adequate drainage. Unfortunately, deep drainage ditches, usually in the form of wooden chutes, are subject to large deformation and do not always function properly, so that the bottom of the cut continues to heave.

Icings frequently occur in cuts and cause numerous complications. They form icefalls which tend to fill the cut. Control of icings is difficult, particularly because it cannot be predicted whether they would form. Figure 64 shows an icing in a railroad cut.

CHAPTER III

SURVEYS AND INVESTIGATIONS FOR ERECTION OF STRUCTURES ON PERMAFROST

A. General Remarks on the Function, Nature, Extent, and Time of the Surveys and Investigations

The previous discussions demonstrate that the best way to prevent structural deformation due to the special conditions existing in permafrost regions is to make a careful study of these conditions and to erect the structure on such a site and in such a manner that the peculiarities of this unique region would be least effective. Accordingly, the most important means to assure stability of structures on permafrost involves careful and detailed surveys and studies of the site for the given structure.

Knowledge and proper consideration of local conditions in design and construction, based on adequate surveys and investigations, constitute the best assurance for structural stability. The main objective of ordinary surveys and investigations is to select a structural site most suitable from engineering and economic points of view under given, actual topographic and geological conditions. In permafrost

regions this task is complicated by the inevitability of considering the permafrost factor which, in cases of a certain combination of ground and hydrogeological conditions, is the decisive factor for structural stability, cost of construction, and future operating expenses. Accordingly, all types and stages of ordinary surveys are necessarily complex when applied in the permafrost region. That is, the usual topographic and geological surveys must be accompanied, particularly in the case of route surveys, by hydrogeological, hydrological, ground, permafrost, and meteorological investigations. This principle of completeness of the investigation must be applied to all stages involved, particularly to the initial stage, that is, during preliminary surveys.

This method of investigation is most important in the case of the permafrost region. Some engineers in this field have the incorrect opinion that the complexity of the surveys, that is, supplementing the engineering investigations with geological, hydrological, and permafrost surveys, should be increased gradually and that the most detailed and complete surveys should be conducted only during the final engineering investigations. This attitude is wrong because permafrost conditions often are the decisive factor, as stated previously.

The permafrost characteristics of the site should be determined first, otherwise later operations and surveys will involve excessive and unnecessary expense. All investigations should be carried out in two stages; the major work in the summer, while doing the supplementary and special studies in the winter. It is best to carry out summer surveys during the period from August to October, and the winter surveys at the end of winter. It has never been and still is not feasible to conduct the basic surveys in the winter becuase of extreme cold, deep freezing of the ground, snow, and swelling of the ground. In addition, many important factors cannot be clarified during the winter.

Some engineers claim that it is preferable to conduct surveys primarily in the winter because it eliminates the difficulties involved in transportation of equipment, food, and feed during the summer in

roadless areas, and because gadflies, mosquitoes, and midges are absent in the winter. This view is incorrect, however. Although it is desirable to avoid difficulties, yet it is impractical to forgo summer surveys because numerous important aspects can be determined only during the summer. These aspects comprise the following: the nature and regime of various streams, the nature of the surface cover, the composition of the ground, the swampiness of marshy places, etc. On the other hand, summer surveys in the permafrost region are insufficient. Some investigations must be carried out in the winter.

B. Surveys and Investigations for Industrial and Public Construction

1. General Instructions

The ultimate objective of all surveys and investigations for construction purposes is to select a site having the most favorable permafrost conditions and simultaneously satisfying the economic and engineering aspects of the given structure. It is obvious that the factors of suitability of the selected site, economic considerations, and engineering expediency must be adequately coordinated after careful evaluation. Accordingly, the dimensions of the structure, its function, pecularities, and relative permanence are involved.

In accordance with the ultimate objective of the surveys and investigations, they should primarily comprise the following: (1) a general study of the locality for the purpose of choosing a number of suitable sites for the given project, (2) general detailed investigations of these sites, and (3) investigation of the particular tracts within the optimum area, on which the largest and most important structures are to be erected. The first two stages are needed for planning and designing the project, while the last stage is needed for working drawings. The degree of detail must correspond to the design stage involved.

All surveys should be conducted in accordance with a preliminary plan of the structure, prepared jointly by technologists and construction engineers, so that investigation of local conditions would yield a relative idea of the dimensions and layout of the projected

structure. If such a plan is not available, use should be made of plans for similar projects.

The aforementioned division of the surveys and investigations into three stages, as well as their particular sequence, is not random. It is based on sound considerations. The first stage constitutes a preliminary or reconnaissance survey with the objective of finding several (two or three) most suitable sites in a given locality within an area of tens of square kilometers and within limits in which relocation would be feasible. The second stage should yield data which would enable one to compare the construction sites obtained during the first survey stage, to select the best site, and to make a detailed and extensive analysis of this site. Choice of the best site is made on the basis of complex and careful analysis of the data obtained, as well as usual economic and engineering considerations. It is obvious that the second stage comprises preliminary studies of the two or three variants involved and definitive studies of the optimum site. Finally, the objective of the third stage is to make a more precise analysis of the data obtained during the first two stages. This stage is applied as needed, in the case of extreme complications or when unforeseen factors occur. In this stage, as in the previous stages, it is necessary carefully to coordinate the local conditions of permafrost and geology with the aspects of production, economics, etc.

As stated above, all investigations must be complete. Therefore, in the case of a fairly large structure, it is desirable that the work involve a group headed by a minimum of the following six persons: (1) a construction engineer who is familiar with permafrost and the given type of structure, (2) a technological engineer who is a specialist in the field of production to which this structure is assigned, (3) a permafrost scientist who is familiar with construction engineering, (4) a geologist who is familiar with permafrost, (5) a hydrogeologist who is familiar with permafrost, and (6) a surveyor.

It is recommended that field studies be made in due time, in two stages--summer and winter. Most studies should be made in the summer, while only special studies that cannot be made in the

summer should be made in the winter. The latter studies comprise the following: snow cover; depth of freezing of various types of ground at various moisture contents and surface covers; location of winter outlet of springs; occurrence of river, spring, and ground icings; thickness of ice; and freezing of rivers, lakes, streams, and reservoirs.

2. General Study of an Area for the Purpose of Selecting a Construction Site

The basic objective of such a study is to select possible variants of construction sites for a given structure. Since it is readily feasible to locate most industrial and public buildings in a given region anywhere within a fairly large distance measuring hundreds of meters of even kilometers, it is desirable to inspect and investigate the largest possible area within this region (an average of 50 sq km). This area is determined from ordinary engineering and economic considerations governing the location of such a project.

The methods used in this preliminary stage may involve visual examination combined with limited drilling and probing whenever feasible. A general study of the locality should yield relative data pertaining to permafrost, geology, topography, and engineering aspects of the area. The exact preliminary and general study of the locality comprises the following steps.

First, it is necessary to study the available material pertaining to the given region, such as books, articles, government publications, etc. Next, it is recommended to make a rapid inspection of the region and to prepare a report containing a general description of the region and including general data about its permafrost, geology, and hydrogeology. The following data are essential: (1) mean annual temperature, (2) thickness of snow cover, and (3) quantity of precipitation (using available sources and introducing corrections when needed). Then, it is required to select two or three most suitable construction sites, taking into consideration the usual factors of size, contour, water supply, and transportation. Finally, it is necessary to compile a detailed description of each of the selected sites. This description should contain the following brief and general data:

(1) nature and amount of moisture in the ground, (2) approximate permafrost level, and (3) occurrence of bedrock.

The following instructions should be followed in selecting construction sites.

a. It is recommended that little consideration be given to areas constituting former river beds, river sediment, marl, or the lowest places in the given region.

b. It is essential to select elevated places because they are least wet and facilitate ready drainage of ground and surface waters.

c. Slopes facing south or southeast should be given preference.

d. It is desirable to select places where bedrock is at the surface or at shallow depth, as well as places where the ground consists of pebbles, gravel, or coarse sand.

e. Places subject to icing processes should be avoided.

f. It is judicious to prefer sites where the permafrost occurs at largest depths.

g. Sites containing imbedded ice or thermokarst lakes are totally unsuitable.

h. Vegetation should be given particular attention. Wooded areas (particularly pine forests) are preferable because such forests are indicative of relative dryness of the ground, relatively deep occurrence of the permafrost, and better engineering properties of the ground.

i. In a region of sporadic permafrost, it is desirable to select a site free from permafrost.

j. In a region of layered permafrost, it is judicious to select a site where the permafrost layer is thin.

k. To avoid complex and expensive flood-control measures, it is necessary to select a site located above the highest level of water in the nearest river, that is, above the level of summer and fall high stages.

3. Detailed General Surveys of the Selected Sites

a. Preliminary Survey

For the purpose of final selection of a specific optimum site, the most satisfactory of the provisional sites should be subjected to preliminary surveys which would yield more accurate and reliable data than those obtained initially. These preliminary surveys should comprise drilling and test pits on a suitable scale. The objective is to determine the following basic characteristics of the site:

1. Nature of the permafrost;

2. Nature and moisture content of the active layer and the upper permafrost layer;

3. Occurrence of bedrock, its properties and stratifications;

4. Occurrence, nature, and position of ground water;

5. Level of the permafrost at several points;

6. Temperature of the upper permafrost layer;

7. Availability and distribution of local building materials.

Comparative analysis of all these data about the various sites involved enables one to decide which site is most suitable, that is, to select the optimum site. It is obvious that the site selected in this way must have the following most favorable external conditions: (1) topography facilitating drainage of surface and ground waters, (2) bedrock occurring nearest the ground surface, (3) lowest ground-water table, and (4) ground having good engineering properties and least moisture content.

At this stage of the investigation, a topographic survey of the locality should be made prior to or simultaneously with other field studies.

b. Final Survey

The objective of the final survey of the selected construction site is to collect all data necessary for drafting the construction plan. Accordingly, this survey must be detailed, careful, and conducted in accordance with the following schedule.

Engineering and geological studies should yield the following data: (1) general engineering and geological description of the given area and the construction site, (2) geological map, (3) longitudinal and lateral geological profiles of the site, determined from test holes and probes, and indicating the upper permafrost limit, and (4) clarification of the previous data about the availability and location of construction materials (sand, gravel, pebbles, etc.).

Permafrost studies aim at clarifying the following aspects: (1) position of the upper permafrost limit and thickness of the active layer at the construction site, (2) nature and properties of the permafrost in the given area and at the construction site, (3) temperature regime of the active layer and the permafrost layer, (4) a map and profile of the ground, and (5) approximate thickness of the permafrost.

Hydrogeological and hydrological surveys should correspond to the following schedule:

1. To determine the hydrogeological conditions of the construction site and the origin of the ground water there, with particular reference to suprapermafrost water;

2. To determine the occurrence and extent of imbedded ice and of ground ice in general;

3. To study the dynamic processes in the active layer, that is, swelling, icings, etc;

4. To investigate streams, springs, and lakes with the objective of determining the feasibility of water supply.

Studies of the engineering properties of the ground should be conducted separately for the active layer and the permafrost. These studies will determine the following: (1) mechanical composition of the ground, (2) moisture content of the ground layers, (3) volume weight of undisturbed frozen ground, (4) specific weight of the ground in its solid phase, (5) thawing tests of permafrost specimens, (6) compressibility of thawed permafrost, (7) experimental adfreezing strength of the active layer and the permafrost, and (8) experimental compressive strength of the permafrost.

Topographic survey of the site constitutes the final survey.

Uniform methods and definite means should be used in all these surveys. In connection with this, only certain explanatory remarks will be presented here, since it is judicious to follow the instructions prepared by the Academy of Sciences SSSR [8].

The geological surveys should comprise the geomorphology of the locality, the history, distribution, petrography, stratigraphy, and tectonics of the bedrock, and analysis of detritus to determine its lithological composition and spatial distribution. Such studies utilize available published material, field observations, and data from test pits and drill holes. These surveys must be made in the summer or fall.

The major aspects in the investigation of permafrost are determination of the upper permafrost limit and the thickness of the active layer. The upper permafrost limit should be determined at various points of the construction site before the frosts set in. These points should be selected relative to the various types of ground, topography of the locality, and vegetation and surface covers, so that two or three controlled drill holes or test pits would be established for each type of ground and surface cover. The number of points for determination of the permafrost level should be adequate to establish the contour of the upper permafrost limit. This is essential because this limit does not always correspond to the contour of the ground surface. Such determination of the upper permafrost limit is only approximate, since obtaining relatively accurate data requires observations over a period of years. On the other hand, use of these data is conditional because, as stated previously, the upper permafrost limit changes considerably when the region involved is developed and structures are erected on the permafrost.

Several methods are used to determine the permafrost level. The simplest are drill holes and test pits. Since drilling does not always yield accurate data, it is preferable to establish test pits wherever feasible, extend them to the permafrost, and then apply drilling. Use may be made of more modern methods, such as the geophysical methods of electrical and seismographical measurements [9, 10];

In the case of merging permafrost, determination of the upper permafrost limit establishes also the thickness of the active layer. When the permafrost occurs at considerable depth, the thickness of the active layer, which in this case corresponds to the depth of winter freezing, should be separately determined at the end of winter. The depth of drill holes generally depends on natural conditions and the size of the structure. Sumgin recommends that drill holes 25 to 30 m deep should be used in the case of large structures. In the case of major construction, it is desirable to drill at least one hole to the lower limit of permafrost.

Test pits and drill holes are useful for investigations other than that of permafrost. Therefore, all test pits and drill holes should be numbered and plotted on a map which indicates their levels. In addition, a log and chart should be kept for each pit and hole, in which a record is made of all the necessary data pertaining to the various investigations.

The nature and type of the permafrost are determined by testing samples of the ground removed from the drill hole and by thermometer measurements. The latter are made during various stages of the drilling operation or after the drill hole has been completed. The first method is preferable, but it requires interruption of drilling in order that the thermal regime in the test hole, disturbed by the drilling operation, could return to normal. This requires a time interval ranging from several hours to three days.

Temperature readings are taken by means of thermometers having special casings. Prior to lowering the thermometer into the test hole, the latter is lined with an ebonite or metal tube. These thermometers are slow recording so that their readings are relatively unaffected by the outside temperature when they are withdrawn from the test hole and read. The bulb in such a slow recording thermometer is enclosed in an insulating material. The temperature should be taken every 40 to 50 cm down to a depth of 2 or 3 m, and every 2 to 3 m at lower depths. Since the temperature measurements are relatively inaccurate, it is desirable to maintain year-round temperature observations

in several test holes on the building site, at depths of 5 to 6 m or at a depth not less than twice the planned foundation depth.

Hydrological and hydrogeological studies should be conducted in two stages--in the summer and winter. It is expedient to utilize the available test pits and drill holes for some of these investigations. The drill holes can be used to measure the ground-water table, which should be done several times in the course of the summer. These measurements would enable one to plot contours of the water table, which would indicate the nature and direction of ground-water flow. The moisture distribution in the active layer and in the permafrost is determined during the summer from samples taken from the test pits at depth intervals of 0.5 m.

Lakes, springs, rivers, and icing sites should be examined in the summer and during the following winter. The study of the winter regime of water sources involves determining the time of freezing and ice formation, the presence of areas which do not freeze, and the winter outlets of springs. The high stages of rivers should be investigated, particularly if they result from atmospheric precipitation.

The problem of water supply under permafrost conditions often is quite complicated and should be given careful consideration. The engineer faces the following two major problems in connection with establishing a water supply in the permafrost zone: (1) finding a sufficiently dependable source of water and (2) bringing the water to the requisite points and distributing it through a piping system.

The complexity of the first problem is evident from the difficulties encountered during construction of the first railroads in the permafrost region. This complexity is due to the fact that most water sources freeze and no water is available in the winter, despite the abundance of water in the summer. Water for numerous railroad lines often has to be obtained by cutting ice and melting it in special containers. Figure 65 shows a bonfire used for obtaining water on the Chichatka River. The water, obtained in such a primitive manner, flowed in channels chopped in the ice and was collected in a tank located in a heated cabin on the ice. A vacuum pump delivered the water

to various points. The second problem is also quite complex, although currently it is usually solved by heating the water. Since the problem of water supply is so important, studies in this field have special significance and are not within the scope of the present work. Therefore, this problem will not be discussed in detail here. It is adequately discussed in the works of M. Ya. Chernyshov [11, 12] . It should be noted, however, that utilization of springs and subpermafrost water is most successful.

Most engineering properties of ground are determined by means of well-known methods that need not be discussed here in detail. It need be pointed out, however, that careful and accurate determination of the volume weight of undisturbed frozen ground is essential in calculations of bearing capacity, settling, etc. Thawing tests of permafrost consist of placing an undisturbed sample of ground taken from the test pit in a glass container and heating it. The behavior and state of the thawed sample are observed and constitute supplementary information regarding the probable behavior of the permafrost during thawing beneath a building. The compaction ratio of the thawed ground is readily determined by means of a device designed by N. A. Tsytovich. All pertinent information is contained in a book by Tsytovich and Sumgin [2, page 376] . This book contains also instructions regarding field tests of the bearing capacity of thawed ground.* The adfreezing strength between the active layer and various foundations is of major importance in many cases. This strength must be determined in the field under natural conditions, since laboratory experiments yield results that are not valid in practice.

Engineer N. I. Bikov designed a practical apparatus for determining the heaving force [13] . Figure 66 shows an improved design of this apparatus having a capacity of 40 tons. The apparatus consists of a conical, reinforced concrete pile 5 m long, which is driven into the permafrost layer by means of an American steam hammer. The reinforcing consists of 6 longitudinal bars 22 mm in diameter and

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* Tsytovich recently proposed a new method for determining the rate of settling of thawed ground. This method involves use of various loads (1 and 3 kg per sq cm, for example).

a spiral bar. A round steel bar 50 mm in diameter, with a washer and nut at its lower end, is inserted into the core of the pile. This bar protrudes 3.3 m from the pile. A wooden or reinforced concrete cylinder, 2 m long and 20 cm in diameter, is mounted over the protruding end. The inside diameter of the cylinder is 7 cm, which is 2 cm larger than the diameter of the steel rod. Upon installation this clearance should be filled with grease, industrial vaseline, or tar. The cylinder is capped by a steel washer with a spring mounted on it. A washer and nut are mounted at the other end of the spring. The spring should be tested and should meet specifications for scale springs. A given force should cause a definite compression of the spring. A pointer, connected to a rack and pinion, is attached to the side of the cylinder. Uplift of the cylinder causes the pointer to rise. While rising, the pointer records the nature and magnitude of the deformation on a graduated drum. The drum is rotated by a gear system actuated by the heaving, the motion being transmitted through the rack and pinion. The rate of heaving is determined from periodical observations.

Bikov designed and used a simpler device for determining the magnitude of the heaving force during tests at a permafrost station. The device is shown diagrammatically in Fig. 67. The short test post is subjected to a load transmitted by a lever of which one end carries a weight P and the other is fastened to a pile driven into the ground to such a depth that heaving cannot force out the pile. Since P is constant, the test requires not less than five installations, each carrying a different load which is either larger or smaller than P. The load P is approximately sufficient to balance the estimated heaving force. Since a large variety of loads would require a large number of installations, use is made of large differences in load. Of course, this reduces the accuracy of the results. Nevertheless, this system is quite simple and can be used under any conditions. A sufficiently large number of installations (10 to 20) would yield fairly reliable data.

Another method (Fig. 68) consists of a lever and an attached weight which are used to withdraw a post located in the active layer and adfrozen to it. This method yields a relative idea of the adfreezing

strength, but the results are rather inaccurate because it is not known whether the testing time is suitable. If the test is conducted early in the season, the results would constitute an underestimation; if the test is conducted when the active layer is completely frozen, the data would constitute an exaggeration because the depth of freezing and, consequently, the area of adfreezing between the post and the ground corresponding to maximum swelling would be less than the entire depth of the active layer.

The compressive strength of the permafrost can be determined by means of methods used in testing the strength of the ground in boreholes. This consists of lowering a shaft carrying a load. Figure 69 shows such a device designed by Bikov. A steel pipe is lowered into the ground until it penetrates into the permafrost; a second pipe of smaller diameter and with a closed end is fitted into the outer pipe and constitutes the shaft. The load is applied by means of a container attached to the shaft.

The topographic surveys are not different from those usually carried out in areas where no permafrost is present; they are simple geodetic surveys which determine the plan and profile of the construction site. These surveys should be done in the summer, since winter surveys may result in major errors due to the fact that the usual swelling of ground in the permafrost region greatly affects the microrelief of the ground so that the readings obtained in the winter differ from those obtained in the summer; in addition, icings and frost mounds occur in the winter. Moreover, winter surveying is difficult even in the southern regions because of low atmospheric temperatures, and is still more difficult in the northern regions where ~~heavy snowfall,~~ snowstorms, and blizzards are a frequent occurrence.

Establishing benchmarks requires special care. They should be established on rocks protruding from the ground. If such rocks are not available, use is made of a permafrost benchmark. It consists of a metal pipe, 5 cm in diameter or larger, with perforations in its lower section. A hole of suitable diameter is drilled in the ground and the pipe is lowered into it (Fig. 70). The depth of this hole, with respect

to the ground surface, should be at least three times the maximum depth of thawing of the upper ground layer. The pipe in this hole is filled with water to the permafrost surface. After this water freezes, the ground is removed within a radius of 0.75 m around the pipe above the permafrost surface, and the pit is filled with gravel or coarse sand topped with a layer of peat.

Another type of benchmark consists of a timber pile driven into the ground by a steam hammer to a depth corresponding to three times the depth of maximal thawing. As in the previous case, the natural ground around the pile is removed and is replaced by non-swelling material. The part of the pile located in the thawing layer should be planed and tarred, and all its cracks should be thoroughly caulked.

In regions of deep freezing, a timber pile may be used as a benchmark if the pile is driven into the ground to thrice the depth of freezing. As in the previous case, the part of the pile within the freezing layer should be planed and tarred. In addition, as described previously, the natural ground around the pile within the active layer should be removed and replaced by well-draining material.

In closing this section, attention is called to the need for considering local construction experience if such is available in the given region. It is essential to make the required special studies in order to ascertain the state of the existing structures. Standard instructions for such studies are given in the appendix.

The standards and specificiations for the design of foundations on permafrost (OST 90032-39) assign great importance to the aspect of surveys and include a number of rational instructions. The following is an excerpt from this manual:

OST NO. 90032-39

III. ENGINEERING AND GEOLOGICAL SURVEYS OF CONSTRUCTION SITES

14. In selecting a building site, particular attention should be given to the type of ground involved. Preference should be given to (a) "dry permafrost" and (b) coarse sand containing no ice wedges and

lenses. It is recommended not to select a building site on ground which is (a) supersaturated, with ice lenses and wedges, or (b) fine-textured, since such ground usually is highly saturated with ice and tends to become fluid upon thawing, which makes it difficult to construct foundations.

15. Engineering, geological, and permafrost surveys of the construction site and nearby area should establish the following:

a. Topography and geological structure of the area;

b. Permafrost phenomena: thickness of the active layer, type of permafrost (layered or continuous, merging or nonmerging), their temperature regime, ect.;

c. Hydrogeological conditions, regime of the ground water (flow direction, velocity, etc), hydromechanical, temperature, and other conditions;

d. State of the ground, occurrence of ice wedges and lenses, mechanical composition, volume and specific weights, moisture content, etc.;

e. Settling of the ground under load, determined by field or laboratory tests.

16. The thickness of the active layer is determined from extensive data of meteorological permafrost stations. If such data are not available, the following must be established:

a. The maximum depth of thawing during the given year in the case of merging permafrost. This is determined at the very beginning of winter freezing (in September or October) by means of test pits or drill holes;

b. The maximum depth of freezing in the case of nonmerging permafrost. This is determined at the beginning of summer thawing (in March or April).

In order to obtain the maximal depth of freezing or thawing, a correction is introduced in accordance with the extensive records of air and ground temperatures, compiled from observations at the nearest meteorological station, taking into consideration the thickness of the snow cover and the time of its formation. If reliable field geological data are not available, it is recommended that the assumed thickness of the active layer be compared with the result of thermodynamic calculations based on local conditions of climate and ground.

Preliminary estimates of the thickness of the active layer may be made in accordance with the data compiled in the following table:

OST Table I

AVERAGE THICKNESS OF ACTIVE LAYER

Permafrost Regions	Thickness in Meters		
	In Sandy Ground	In Clayey Ground	In Ground with Peat
In areas south of the 55th parallel	3.0 to 4.0	1.8 to 2.5	0.7 to 1.0
At the latitude of Yakutsk (62°N.)	2.0 to 2.5	1.5 to 2.0	0.5 to 0.8
Along the coast of the Arctic Ocean	1.2 to 1.6	0.7 to 1.0	0.2 to 0.4

C. Surveys and Investigations for Road Construction

1. General Remarks

The basic objective of ordinary engineering surveys is to select the most suitable route for a given projected line with respect to actual topographic and geological conditions. This objective is valid also in the case of permafrost conditions, but its attainment is greatly complicated by the additional factor of permafrost. In some cases, particularly when the effects of given ground and hydrogeology are cumulative, permafrost conditions tend to be the decisive factor in the stability of earth roadbeds and other road structures, in so far as costs of construction and future operation are concerned. Therefore, all aspects and stages of engineering surveys should be complete and conducted simultaneously with other studies necessary to clarify all aspects of the occurrence of permafrost in the region of the projected line.

The topographic route surveys should be accompanied by geological, hydrogeological, ground, permafrost, and meteorological studies of the route involved. It is preferable to conduct all these engineering surveys in the summer because topographic and hydrogeological conditions can be evaluated more fully and correctly and because the work involved is cheaper and more rapid (particularly in regions with thick snow cover and intense snowstorms). Investigations relating to frost, hydrology, and hydrogeology should be conducted in two stages—in the summer and at the end of winter. Winter investigations comprise the following aspects: snow cover; depth of freezing of various types of ground at various moisture contents and surface covers; winter outlets of springs; icings (river, spring, and ground); thickness of ice; extent of freezing of rivers, lakes, other watercourses, and bodies of water.

All winter investigations should be conducted during the first two stages of the engineering surveys. Winter investigations are permissible during the final surveys only if carried out on a small scale and when the data obtained during the reconnaissance survey are inadequate to solve the special problems involved. The relative merits of a given season for surveying operations have been previously discussed in detail.

The usual personnel of surveying parties is determined by general engineering considerations. In the case of complete surveys in permafrost regions, however, it is judicious that this personnel include the following specialists: geologists who are thoroughly familiar with the aspects of hydrogeology and soil science, as well as geophysicists who are specialists in the field of permafrost science. The chief of the party or of a given team should have adequate theoretical and practical experience in the fields of geology, hydrogeology, and frost science, so that the results of the geological, hydrogeological, and permafrost investigations would be properly evaluated, coordinated, and utilized. It really is very important that the party chief qualify as a frost scientist because errors that could not readily be corrected have resulted from the fact that the engineers in charge had inadequate knowledge of permafrost and of the complex of problems related to permafrost.

2. General Nature, Sequence, and Elements of These Surveys

To obtain more reliable data pertaining to the route and construction conditions, the following sequence of surveys is recommended:

(1) Complex reconnaissance.

(2) Preliminary complete instrument surveys.

(3) Final complex surveys.

All these surveys must be conducted so as to meet the general engineering requirements for road planning, but the uniqueness of the permafrost region involved should be taken into consideration in accordance with the following discussion.

To assure continuous progress of the engineering surveys, particular attention should be paid to the complex field reconnaisance which should cover the widest possible area along each of the provisional routes of the projected line. The width of the belt involved is determined by local topographic, geomorphological, and permafrost conditions, but should average about 5 to 10 km. The prime objective of the complex field reconnaissance (the first stage of the engineering survey) is to select various probable routes, including those most favorable with respect to permafrost. Correct solution of this problem would preclude subsequent waste of time and money during the

preliminary and final instrument surveys, since such a solution would eliminate those variants which are regarded as unsatisfactory because of permafrost considerations.

In accordance with its main objective, the complex field reconnaissance furnishes general geological information about the region (tectonics, morphology, stratigraphy, and lithology) and clarifies the aspects characterizing the properties of the permafrost within the region of each projected variant. These aspects comprise: type of ground and moisture content; hydrology; surface cover; occurrence of mari, icings, and imbedded ice; thickness of active layer; position of upper permafrost limit; thickness of permafrost (within practical limits); type and temperature of the permafrost; and thickness of ice in and occurrence of complete freezing of rivers and other bodies of water. In addition, data are obtained regarding the meteorological aspects of the region and other physical and geographical factors.

The preliminary instrument surveys comprise general, detailed and accurate topographic, geological, and permafrost investigations of the two or three routes involved. In addition, the geological and permafrost data, obtained during the reconnaissance survey for the purpose of comparing the various routes, are supplemented and rendered more precise if the results of the reconnaissance survey were insufficient to select a definite route. The main objective of the complex preliminary engineering surveys is not to compare the various routes involved, since this is basically accomplished during the reconnaissance survey, but to improve the selected route by means of partial variants. It is obvious, therefore, that all these investigations are carried out on a narrower belt than that involved during the reconnaissance survey, but with greater precision and utilization of test holes. Test pits and drill holes are made at all points characterizing changes in the contour of the upper permafrost limit. Such points comprise mounds, depressions, banks of rivers and other watercourses, swamps, and places where changes occur in the type of ground, surface cover, slope exposure, etc.

The objective of the final complex surveys is to adjust and fix the route along the line established during the preliminary surveys.

During this surveying stage, the position of the roadbed axis is adjusted and the most advantageous longitudinal profile is determined with respect to permafrost conditions. These surveys furnish all of the data necessary to prepare separate plans for each structure and to determine the special measures which would assure stability of the roadbed and other structures under the particular permafrost conditions existing in each given case. It is obvious, therefore, that the complex investigations involved in this stage of engineering surveys relate only to those particular areas along the route for which the data obtained during the previous surveying stages were inadequate because of the particular complexity involved. The investigations cover a still narrower belt than that involved during the preliminary surveys, but they are more complete and precise.

Such investigations should be made in cuts (except when the cuts occur in solid rock, stones, gravel, or coarse rubble), in swamps, and within boundaries of earth fills if their height is less than the thickness of the active layer in the given locality. In addition, such investigations should be carried out at the sites of stations and within the boundaries of road structures and large structures such as depots, roundhouses, shops, pump houses, power stations, etc. These investigations should yield data which would facilitate preparation of longitudinal and transverse geological profiles. These profiles should show the stratification and material of the active layer and permafrost, the natural moisture content at various depths, and the distribution and size of ice lenses.

The final complex surveys should yield exhaustive data regarding the geology, hydrogeology, type of ground, and permafrost conditions of the given section of the projected route. These data should be adequate for preparation of construction plans.

3. Aspects and Nature of the Geological, Hydrogeological, Permafrost, and Hydrological Surveys

The geological and hydrogeological surveys comprise the following two objectives: (1) to obtain the ordinary data necessary for selecting a route that would assure stability and most economical construction of the

roadbed and other structures, and (2) to obtain special data which would facilitate the most satisfactory clarification of all the properties of the permafrost in the region involved. The extent and nature of these investigations during the reconnaissance and preliminary stages are determined in accordance with the instructions in Chapter III-B. The geological investigations during the final surveys should provide the following:

1. A geological map of the area along the projected route.

2. A general engineering and geological description of the region.

3. A detailed and sectional geological description of the route, indicating the depth of bedrock. It should include geological profiles of the sites where major buildings are planned. It should show the topography and nature of the locality, vegetation, surface cover, as well as the type of ground and moisture content within the active layer, the talik, and the permafrost.

4. Information about the physical and mechanical properties of the ground.

5. Data pertaining to the level and regime of suprapermafrost, intrapermafrost, and subpermafrost waters.

6. A detailed engineering and geological description of major siding areas.

7. An extensive investigation of slopes, particularly if they are unstable, including longitudinal and transverse geological profiles. These profiles should extend from the foot of the slope to the crest, or to the point where the slope changes into a flat terrace.

8. A detailed description of the sources of construction and ballast materials (stone, gravel, pebble, sand, rubble, clay, limestone, and gypsum), indicating quantities and location relative to the projected line. In the case of permafrost regions and mountainous areas with narrow valleys in the southern portion of the Far East, it is advisable to search for construction and ballast materials on the plateaus along both sides of the valley. Here old river deposits of sand, gravel, and pebbles often are found even at altitudes of 200 to 300 m above the valley floor.

The permafrost studies include clarification of all those aspects of the occurrence of permafrost in the region of the projected railroad that may affect the choice of a given route, the design of the roadbed and other structures, and the use of special measures (drainage, etc.) to assure the stability and strength of the roadbed and other structures erected on the permafrost. Accordingly, the following work is involved:

1. Determination of the nature of the permafrost (areal continuity, sporadic, island, vertical continuity, layered, imbedded ice).
2. Determination of the upper permafrost limit and the thickness of the active layer.
3. Determination of the thickness of permafrost at particular points.
4. Compilation of a map of permafrost phenomena.
5. Determination of the areal location of taliks and permafrost islands.
6. Determination of the extent of ice lenses, in plan and profile.
7. Determination of the temperature of the active layer and permafrost at various depths.

All these geological, hydrogeological, and permafrost data can be obtained by means of surveys within a wide area along the route, as well as by means of drill holes, test pits, electrical measurements, other suitable field methods of geological investigations, and, finally, by means of photography. It is advisable to photograph typical or particularly complicated geological phenomena which cannot be readily described.

The number, depth, and location of drill holes and test pits depend on the following factors: local geological conditions, nature of the permafrost (continuous, containing taliks, sporadic, layered, etc.), type of ground, type of structure (fill, cut, culvert, bridge, building, etc.), and the stage of the complex surveys (reconnaissance, preliminary, final). It would generally be quite advantageous to prepare a special set of instructions for investigations under such conditions.

Meanwhile use can be made of the manual mentioned previously, prepared by the Academy of Sciences. If existing structures occur along the route, it is essential to examine them in accordance with the instructions presented in the appendix.

The objective of the hydrological and hydrogeological investigations is to obtain data pertaining to the number and location of water passages, the protection of the roadbed and other structures against surface water, and the problem of water supply. In addition, the hydrogeological investigations should yield data which would clarify all the aspects of permafrost in the given region, as well as the related physical and dynamic phenomena in the active layer and on its surface. To obtain these data it is necessary to conduct the following supplementary investigations, in addition to the investigations involved in the usual engineering surveys:

1. To determine the winter regime of rivers and the depth of freezing of streams, as well as the duration of such freezing and whether water continues to flow beneath the beds of streams frozen to the bottom. This information may be obtained from local people, if any, but must be verified by winter tests;

2. To determine the general regime of surface water in the summer and winter;

3. To determine the occurrence of river icings, their origin, and possible effect on the routing of the projected line;

4. To determine the quality and quantity of water and available water supplies with regard to suitability for drinking purposes, locomotives, and construction;

5. To determine the degree of aggressiveness of ground water, so that suitable measures could be taken to protect the masonry and concrete parts of the structure;

6. To study runoff conditions.

After office preparation of the data obtained, it is essential to compile the following reports, in addition to those normally required for preparation of engineering drawings and those required by various government agencies:

a. Geological maps showing permafrost phenomena;

b. A topographical map;

c. A description of the route, comprising information about the aspects of engineering geology, hydrogeology, permafrost, and ground;

d. A description of the sources of construction and ballast materials.

e. A geological profile of the route, giving data about permafrost and ground;

f. Tables of test pit and drill hole measurements, containing data about the permafrost;

g. Reports and data pertaining to investigation of existing structures and prepared in accordance with the special instructions presented in the appendix.

4. Special Instructions Regarding Rational Routing of the Road in Both Plan and Profile

The fundamental principle underlying the following instructions regarding planning of the right-of-way is proper and skillful selection of the route. The factors influencing such selection are the conditions of permafrost, geology, and hydrogeology, as well as usual economic and engineering conditions. These instructions are of exceptional importance because the major steps to control the harmful effects of permafrost and its inherent complications should be taken before the beginning of construction, during the period of surveys and planning when choice of proper route and design can assure strength and stability of the roadbed and other structures at minimum cost.

The type of ground and moisture content are decisive as far as choice of the most stable route is concerned. Therefore, it is essential to select areas in which the ground would be least affected by permafrost phenomena. Accordingly, solid rock is the most suitable ground because permafrost has no harmful effect on solid rock (with respect to construction). Permafrost has very little effect on disintegrating rock and on bouldery talus if the cracks in the former and the spaces between individual boulders in the latter are not filled with fine-textured material. Of course, economic considerations always are an important factor.

In laying out a road under ordinary conditions, that is, outside the permafrost region, surveyors endeavor to avoid rocky ground because it increases both the cost and duration of construction. Some surveyors tend mechanically to apply this rule, derived from accumulated experience, to permafrost regions without taking into consideration the properties of permafrost and introducing the necessary corrections. Yet one of the most important rules in laying out a highway or railway under permafrost conditions is to select a route traversing the driest areas containing coarse-textured ground (coarse sand, gravel, pebbles, rubble) or, better yet, solid rock. Such a route assures considerable stability of the roadbed and structures, makes construction easier and cheaper, and reduces maintenance costs.

In laying out a road, therefore, it is essential to avoid areas where the ground consists of fine sand and sandy loam, and particularly silt, because these materials do not assure stability of the structures. When comparing variants of routes in permafrost regions on rocky and soft ground, the following should be taken into consideration:

1. Fills and cuts in rocky ground are absolutely stable and do not require any special measures or limitation in the height of fills or depth of cuts.

2. The size of fills and cuts in rocky ground is considerably smaller than in soft ground because the slopes can be steeper and the roadbed can be narrower. Depending on the nature of the rock, the permissible slopes in rocky ground range from 1.0:0.5 to 1.0:1.1 in the case of cuts and from 1.0:1.25 to 1.0:0.75 in the case of fills, while the corresponding slopes in soft ground in the permafrost region range respectively from 1:2 to 1:5 and from 1.0:1.5 to 1.0:2.5, or maybe even shallower. It may even be necessary to resort to other costly measures which still do not assure stability of the structure.

3. It is not necessary to stabilize the slopes of rocky cuts and fills, while such stabilization is most complicated in the case of soft ground in the permafrost region.

4. Drainage installations can be smaller and of simpler construction than in the case of soft ground.

5. Use of rocky ground is additionally advantageous because

it is not necessary to establish any quarries in order to obtain material for concrete aggregate; for masonry of various structures; for riprapping slopes, beds, ditches, and other drainage installations; and for use as ballast.

6. Use of rocky ground eliminates the need for complicated, often unreliable and very costly special measures to assure stability of the roadbed, which are essential in the case of soft ground.

7. The effects of disturbed permafrost regime on the roadbed, due to construction of the roadbed, are negligible in the case of rocky ground.

8. Cuts in permafrost require the use of explosives, so that they are as costly as cuts in rock. It is possible to make cuts in permafrost using gradual thawing with solar heat or artificial fires, but such operations are more costly than blasting of cuts in rocks because the process is complicated, not productive, and very slow.

9. The soft ground excavated from cuts in frozen ground often is unsuitable for use in fills, which increases the distance and volume of haulage involved.

10. Proper construction procedures prevent the occurrence of swelling in cuts made in rocky ground, while most careful work cannot entirely prevent the occurrence of swelling in cuts made in soft ground. It is known that control of swelling in permafrost is most difficult.

In many permafrost regions, such as the Far East, valleys and watersheds constitute the wettest areas, while slopes, particularly those facing south, are the driest areas. Bedrock occurring close to or at the surface is found on such slopes more often than in valleys or watersheds. Therefore, it is advisable to route the road along the slopes and, in comparing variants, to consider the advantages inherent in slopes facing south. This reference to the Far East indicates that the statements pertain to the region of the Far East and to other regions having analogous geographical and geophysical conditions. It is obvious that in the case of other regions it is necessary to take into consideration the characteristics of

these regions. In the north of European USSR, for example, river and stream valleys and low regions in general constitute areas free of permafrost or where permafrost occurs at a depth of more than 5 m. In this region, therefore, permafrost considerations make it preferable to route the road along river valleys and stream shores, avoiding slopes and watersheds. The north of Asiatic USSR has its own pecularities. Here constant strong winds blow the snow off elevations and windward slopes and cause it to accumulate in depressions and on leeward slopes. Accordingly, other conditions being equal, the upper permafrost limit occurs at a greater depth on the leeward slopes than on the windward slopes. However, it is not judicious to select a route on the basis of this factor alone, without considering the other factors involved. Since the winds during blizzards in this region reach a velocity of 20 to 40 m per sec, it may be more difficult to check snowdrifts than to counteract the harmful effect of the permafrost. This factor should be taken into consideration during route surveys. Areas where snowdrifts occur should be avoided even if it involves routing the road along slopes with shallower permafrost level and less satisfactory ground material. Similarly, in the case of subarctic regions where the summer sun does not rise high above the horizon and completes a nearly circular orbit around it, the north, south, east, and west slopes are heated nearly equally. Therefore, it is no longer essential to route the road preferably along sunny slopes.

The foregoing discussion reemphasizes the basic concept of this text that negative effects of permafrost can be counteracted only by means of exhaustive studies of all local conditions in the course of the surveys and by means of a series of measures adopted during construction. Accordingly, the entire Chapter III (particularly section C-4) aims at emphasizing the fact that, in order to obtain a construction site or route on which the structure or road would be most stable, it is necessary to consider carefully all local specific properties and all probable maintenance problems, while making a thorough engineering and economic analysis of all data.

In laying out a road, it is usually judicious to avoid areas containing imbedded ice or ice lenses, as well as areas in which river, ground, or spring icings occur. Such areas are particularly unsuitable

locations for either large or small depots and stations. Large stations should be located on areas where the bedrock is nearest the surface (not more than 5 m deep).

With regard to the longitudinal profile of the route, it is advisable (not imperative) to lay out the road in such a way that would involve use of few cuts in frozen ground and relatively low fills, 2 to 5 m high, in the case of ground consisting of medium sand, fine sand, or loam. Use of cuts in permafrost consisting of silty material should definitely be avoided. Unless the material involved is stone, gravel, or coarse sand, the height of fills should be not less than 2 m in order to avoid swelling which may occur in a low fill due to saturation by capillary action.

If the permafrost level is high and the active layer is excessively wet, fills of earth material should be not more than 5 m high because their slopes tend to slide. The height of a fill in such areas should be not greater than the thickness of the active layer involved. Moreover, the permafrost beneath a high fill tends to rise and if special measures are not taken, penetrate into the fill itself in th form of a longitudinal hump with steep side slopes along which the side of the fill may slide and cause complete deformation of the fill. If t height of the fill does not exceed the thickness of the active layer in volved, the upper permafrost limit does not rise above the natural grou surface. Fills consisting of rock and boulders may be of any height be cause such material drains well and the fills do not become saturated w water, do not swell, are not subject to solifluction, and do not disint grate upon settling. Accordingly, no limit is imposed on the height of such fills.

Cuts in rocky or stony ground may be of any desired depth. If it is impossible to avoid cuts in frozen earth material, the road should be laid out in such a way that the bottom of the cut would not penetrate into the permafrost or even approach the upper permafrost limit. In the case of a thick permafrost layer occurring deep below the surface, the bottom of the cut should be located above the upper permafrost limit a distance equal to the depth of summer thawing in th

given area. If the permafrost layer is relatively thin, so that such an arrangement is not feasible, it is necessary to lay out the road in such a way that the bottom of the cut would be below the upper permafrost limit. These last two requirements are motivated by the fact that the bottom of the cut should be stable, and should neither settle nor swell. In many cases it is more advantageous to relocate the line in order to satisfy the requirements of stability than to take special measures for stabilizing the bottom of the cut. Of course, this involves careful consideration of all factors, including the engineering and economic aspects of the problem.

The extent of deformation of cuts has been amply illustrated and described previously. Another example of the difficulties which may arise in such cuts is shown in Fig. 63 and described in the corresponding text. The description indicates that it was necessary ultimately to abandon the cut and route the line along its lower shoulder.

In the case of marl, it is recommended to use only fills because excavation of cuts is extremely difficult. The rare cases of shallow marl overlying rock on slopes or in narrow watersheds constitute an exception.

It is essential to avoid laying out a road over imbedded ice (ice lenses), the thawing of which may cause considerable settling of the natural ground surface. This is particularly valid in the case of thick layers of imbedded ice or extensive lenses. In special cases in which the projected line is subject to such conditions, use should be made of fills only. This refers to cases where the imbedded ice occurs at a relatively shallow depth (several meters) and where the thickness of the ground layer overlying the ice tends to conserve the ice. In this case, melting of the ice and settling of the ground would result from mere removal of the moss or vegetation cover. It is readily understood, therefore, that cuts cannot be used. The fill should be sufficiently high to assure conservation of the ice. The topography of imbedded ice usually makes such an arrangement feasible.

It is advisable to locate railroad stops on shallow slopes, preferably those facing south, and primarily on fills not over 2 m high. In

the case of rocky or stony ground, however, such stops may be located in cuts or on areas consisting of both cuts and fills.

CHAPTER IV

PRINCIPLES AND ASPECTS OF STABLE CONSTRUCTION

A. Construction Methods and Their Application

Erection of any kind of structures and development of new regions result in disturbance of the natural and historical regimes of the permafrost. The permafrost may vanish, recede, remain constant, or even rise and approach the ground surface.

A change in position or state of the permafrost is of great importance in construction because, as previously established, thawing of the permafrost at the base of a structure erected on frozen ground causes deformation and even collapse of the structure. It is essential, therefore, to make advance analysis of the probable stability of the structure under conditions that would exist after construction has been completed. On the other hand, the fact that the upper permafrost limit may rise or fall, depending on existing external conditions, indicates that it is feasible to some extent to control the position of this limit or to maintain it within certain bounds with respect to its normal position. The mechanical strength of permafrost in its frozen state is very high, so that it is obviously desirable to utilize the permafrost as a base for structures.

Accordingly, it is necessary to develop special construction methods for the permafrost region. These methods are designed to utiliz the positive properties of the permafrost, and to take into consideratio its peculiarities and phenomena which result in conditions causing struc tural deformation. OST Manual No. 90032-39 recommends the following tw construction methods, or two methods of foundation design. These method depend on the geological, hydrogeological, climatic, and permafrost conditions of the given area, as well as on the nature of the construction site and the temperature regime and design of the structures involved.

1. The first method, method A or the passive method, involves conservation of the permafrost at the base of the structure.

2. The second method, method B or the active method, allows for disturbance of the frozen state of ground at the base, but makes provisions in the structural design to compensate for settling during thawing.

The advantage of the first method is unquestionable in all cases where conservation of the permafrost can be assured. This method is applicable primarily in the northern permafrost regions but may be used in the southern regions as well. At Skovorodino, for example, it proved feasible to conserve the permafrost and even raise its upper limit beneath several wooden buildings. Except in the case of heating plants, boiler houses, and other structures radiating large quantities of heat, the passive construction method may be applied to all types of buildings if the permafrost is thick and stable and if the temperature of the ground at the design level of the foundation base is not above -0.5° C, provided a ventilated air space is used.

It is theoretically feasible to conserve the permafrost beneath buildings radiating much heat. In practice, however, such construction is complicated and costly because it involves arrangement of a ventilated air space which extremely complicates the design of such structures, and because it involves a series of special insulating operations. The need for a ventilated air space imposes considerable limitations upon the passive method of construction; however, an air space is inevitable in common construction.

It was explained in Chapter II that thawing of the permafrost at the base of a structure is due to the fact that both the foundations and the entire mass of the structure radiate heat through the floor to the permafrost. The latter source of heat may be more effective than the former. Heat transfer into the ground may be quite extensive, particularly if the floor of the building rests directly on the ground, and cannot be counteracted by the cold of the air because the ground beneath the building is protected by the building itself. It is obvious that under such conditions the heat may penetrate quite deep into the ground.

According to engineer V. A. Byalinitsky, a certain multilevel railway depot radiates 2,700,000 kg-cal, or 130 cal per sq m of floor space, into the ground from each level every 24 hr. The permafrost beneath this building thawed to a depth of 10 m. Byalinitsky correctly remarks that use of slag, peat, or other insulating layers would be of no value because any insulation would only retard the thawing but would not reduce its extent.

The situation would be quite different if the heat radiated through the floor could be channeled away before it reached the ground. A ventilated air space has precisely this effect. Such an air space makes it possible to maintain, beneath the floor of the building, a winter temperature differing little from the temperature of the outside air. The summer temperature in the air space is positive, but it is lower than the outside temperature. In this case, less heat and almost as much cold penetrates into the ground beneath the building. As a result, the upper permafrost limit may rise, which actually happened in the case of a number of buildings at Skovorodino.

Conservation of permafrost beneath buildings can be achieved by means of artificial freezing, but the engineering and economic aspects of this method have not been sufficiently developed as yet, although use of this method seems quite suitable in some cases. It would be valuable and timely if designers investigated these problems and developed some practical methods. M. I. Sumgin, M. M. Krylov, and N. G. Trupak have advanced some particularly interesting ideas and recommendations in this field [14]. The concept of "thick layer" of permafrost, which is a requisite for applying the passive method of construction, is not well defined. However, taking into consideration the thickness needed to sink foundations into permafrost and the possibility that the upper portion of this layer would thaw after an area has been developed, it may be assumed that a layer 8 to 10 m thick is approximately adequate, although a layer 15 to 20 m thick should be the criterion in the case of large structures. The allowable minimum negative temperature of the permafrost, specified previously as -0.5° C, is strictly relative and has no theoretical basis. It should be noted,

however, that relatively extensive plastic deformation of the frozen base may occur, and the permafrost would not be stable and would readily thaw at temperatures above -0.5° C.

Despite the fact that the passive construction method is recommended by the OST Manual and despite the preceding discussion, it should be noted that this method has been verified in practice over a relatively long period of time only in the case of small wooden structures. The behavior and stability of large masonry structures with ventilated air spaces have not yet been fully determined in practice, although theoretical considerations lead to the belief that the passive method is entirely feasible in the case of such structures also.

It is probable that the second construction method can be applied in a few cases where settling is small and the structures are of special design, since item 26 of OST Manual 90032-39 specified that this method should be used in the case of "structures so designed that they may reasonably well withstand uneven settling resulting from non-uniform thawing of the ground beneath various parts of the building."

Most ordinary brick structures cannot withstand considerable settling, and corresponding improvement in their design necessitates special and costly arrangements. The few attempts along this line were costly and proved unsatisfactory. Frame structures cannot readily be designed to withstand settling, except in the case of steel frames and when it is feasible to regulate the height of the columns, which is also costly and difficult. Moreover, automatic regulation of the elevation of frame elements is feasible only within relatively narrow limits, while settling due to thawing of the ground usually is considerable.

In spite of the fact that the shapes of such structures have not been worked out as yet and have not been tested, it is necessary to consider the fact that such structures are theoretically feasible. This is corroborated by the fact that buildings, designed to withstand considerable settling, have been erected on loess and proved satisfactory [15, page 513]. It is true that the conditions differed from

those existing in the case of permafrost, but the example is pertinent because of similarity with respect to considerable settling. Accordingly, the second construction method, which is more theoretical than practical, will probably find relatively limited application. Satisfactory examples of practical application of this method are not available as yet, even in the case of small structures, except for a few buildings on crib foundations which are no longer popular because their stability is uncertain. It is difficult to recommend this method for relatively important structures. It is preferable to avoid this type of design, except in cases where the ground is not supersaturated or does not contain more than 30 per cent moisture by weight, and consists of coarse sand, gravel, or pebbles. This method cannot be recommended for ground consisting of fine-textured material, unless special investigations, tests, and calculations prove that settling will be slight and will not cause excessive stresses in the structure. This is p ssible only if the aforementioned types of ground contain little moisture.

Construction of a certain electrical power station is an example of the feasibility of this method in the case of sandy ground containing little moisture. The boiler room was built in 1930. Investigations and tests showed that the site of the projected boiler room was underlain by a layer of frozen sand at a depth of about 5 m and possessing good construction properties. Accordingly, the foundation was designed in the form of columns resting on a continuous reinforced concrete slab 35 cm thick placed on a thin layer of tamped gravel. The slab was covered with well-tamped sandy loam and a layer of sand. Until 1938 the building had not deformed. It is not known whether the frozen base has thawed, but it should be assumed theoretically that thawing did occur.

It is advisable to metion here another (a third) construction method, although the OST Manual does not refer to any third method. This method involves artificial elimination of the entire permafrost or part of it prior to construction. The extent to which the permafrost is eliminated is such that construction can proceed as if on ordinary thawed ground. It is obvious that this method, like the preceding two

methods, is feasible only under overall favorable conditions. It should be noted that this method has not been applied in organized and well-planned operations. However, a certain amount of indirect data pertaining to this method are available, and this method is theoretically feasible and even advisable.

Construction preceded by elimination of the permafrost is both economical and feasible in cases where the temperature of the permafrost is near zero, as well as in cases of layered permafrost consisting of thin layers, sporadic permafrost, and providing excess moisture can be removed from the ground. The permafrost can be eliminated in some cases by means of preconstruction thawing utilizing solar heat, which involves removing the surface cover and exposing the surface, as well as by means of melioration and drainage. The permafrost can be eliminated in other cases by means of steam injections provided that it is possible subsequently to remove from the thawed layer both the original ground water and the water condensed from the steam. Erecting a structure might be feasible after the ground has thawed to the required depth, has settled, and the water flowed away or has been removed from the ground. This method is hardly feasible in the case of fine-textured ground which is supersaturated and does not readily yield the water.

There are particular cases where construction of foundations is feasible regardless of the permafrost. One such case is when frozen or unfrozen solid rock or, if the permafrost layer is thin, even good thawed ground having high bearing capacity occurs at a depth of 8 to 12 m below the surface. In erecting an important structure under such conditions, it is advisable to avoid shallow foundations on permafrost and to utilize the underlying rock or thawed layer as a base. The foundations are erected on sunk wells which are prepared by means of steam points. Figure 71 is a diagram of such an arrangement.

This type of design was used on one project. The conditions involved were as follows. The building was to be located on a floodplain. The area was a mar with sparse scrub. The geological section was as follows: a 20 to 30 cm vegetation layer underlain by a 1.15 to 1.[illegible] cm layer of supersaturated very fine yellow sand containing ice

crystals; beneath this layer was a 5.0 to 5.5 m sand layer of various grain size, saturated with ice and containing a small amount of gravel; the lowest layer, occurring at a depth of 7.0 to 7.5 m below the ground surface, consisted of highly fissured hornblende rock. A fill was deposited over the entire area. Its thickness at the building site was 1.5 m. It was decided to sink the foundations to bedrock. The foundations were designed in the form of sunk wells. It was proposed to sink these wells by steaming the permafrost with steam points and subsequently removing the thawed liquid ground by a suction dredge.

Under analogous conditions it would be quite suitable to arrange the foundations on piles driven to bedrock (Fig. 72). This is much cheaper than sunk wells. The piles may be timber, reinforced concrete, or steel, depending upon local conditions. The tubular steel piles, widely used in the United States, are highly recommended for use in cases similar to that under consideration, that is, when the pile has to pass through an unsuitable layer of weak ground and rest on bedrock. These piles are hollow steel pipes with closed ends, driven to the bedrock. The ground is removed from the pipe by washing or compressed air [16], and then the pile is filled with concrete. The pile diameter ranges from 25 to 50 cm. The wall thickness ranges between 7 and 15 mm. The maximum length of a pile is 30 m. The pile is made up of standard pipe sections 7 m long. These steel pipes, filled with concrete, have a high load capacity. American standards permit a load of 121 tons per pile resting on bedrock.

Figure 73 shows pile-driving operations on building construction in America. Numerous cylindrical piles readied for driving are seen in the foreground near the group of upright piles. It is of interest to note that a certain bridge with ten spans of 12 m each is resting on steel pipes (d = 20 cm) filled with concrete. The piles are driven into the river bed to bedrock.

The following two major causes of deformation of various structures were established in Chapter II: (1) thawing of the permafrost at the base, and (2) swelling of the active layer. All of the construction methods discussed previously were designed to counteract

the first cause of structural deformation, or thawing, and are not related to the aspect of swelling. Accordingly, whichever construction method is used, it is necessary to take into consideration the swelling of the active layer and to take proper measures to avoid deformation due to swelling. These measures will be discussed in Chapter IV-B.

The possibility and feasibility of using a given construction method should be established in each concrete case after careful study of the building site with respect to local permafrost conditions and nature of the structure, and after careful analysis of all economic and engineering aspects involved. The following excerpts pertaining to construction methods are taken from OST manual 90032-39.

OST No. 90032-39

V. PRINCIPLES OF DESIGN OF BASES AND FOUNDATIONS

21. In designing foundations, it is necessary to consider the changes in ground conditions and permafrost regime which result from development and utilization of the area and which usually cause lowering of the upper permafrost limit. This involves consideration of the effect upon ground temperature due to the structures themselves as well as other surface and subsurface factors such as water mains, canalization, destruction of vegetation and snow covers, grading, drainage ditches, etc.

22. One of the following two methods of foundation design may be used, depending upon geological, hydrogeological, climatic, and permafrost conditions of the area involved, as well as the nature of the construction site, and the temperature regime and design aspect of the structures:

Method A, or the passive method, involves conservation of the permafrost near and beneath the structure;

Method B, or the active method, involves disturbance of the permafrost near and beneath the structure, but adapts the design to allow for settling during thawing.

23. Conservation of permafrost may be assumed if the permafrost is thick, continuous, and has a low temperature, and if the structure does not radiate much heat. In this case the design may be based on method A. In the case of workshops radiating much heat, the use of method A requires special measures for permafrost conservation.

24. It is difficult to conserve the permafrost beneath heated buildings if the permafrost is thin, sporadic or layered, and has a temperature near zero. In this case, therefore, the design is based on the assumption that the permafrost beneath the building will gradually thaw and that uneven settling of the foundations will result, so that method B is suitable.

25. The following steps are recommended in the case of a structure erected in accordance with method A:

(a) To provide an air space beneath the floor, ventilated in the winter and enclosed in the summer, with proper insulation of the skirting and creosoting of the wooden parts. This space should be at least 0.5 m high. In the case of important structures, this height should be checked by means of thermodynamic computations.

(b) To build the foundations in the form of separate supports (columns, footings, etc.) sunk into the permafrost and having a minimal cross section satisfying load and design conditions.

(c) To use foundation materials of lowest heat conductivity and to apply insulating layers, so as to reduce the transfer of heat to the subgrade.

(d) To cover the ground surface near and beneath the building with a protective layer of material that does not conduct heat (ash, slag, peat, etc.) and to construct dikes and troughs to divert the surface water from the structure.

(e) To insulate the underground steam, sewage, and water mains near the building, so as to eliminate their effect on the permafrost beneath the building.

(f) To discharge the industrial as well as household water far from the building, so that the water would not seep into the ground near the foundations.

(g) To make all foundation excavations when the ambient temperature is below zero, so that the foundations would be placed on frozen ground. Prefabricated foundations, requiring only assembly, are preferable.

26. Method B is used when the design of the structures allows for nonuniform settling that may result from uneven thawing of the ground beneath various parts of the structure. The probable extent of settling is determined from tests on the permafrost of the foundation base. These tests should be conducted in accordance with the specifications given in sections III and IV of the OST specifications.

27. When method B is used, the thawing process should be retarded in order to obtain more uniform thawing of the permafrost beneath separate parts of the structure. This retardation is accomplished by means of the measures presented in item 25 above.

28. Various methods can be used to decrease the nonuniformity of settling. They are as follows: placing sand mats beneath the foundations, using grillage foundations or foundations with continuous concrete or reinforced concrete slabs, using piles, etc.

29. In order to reduce the effect on nonuniform settling upon buildings, it is advisable to use a design that would permit jacking up of individual parts that have settled.

30. Method B should be used in the case of ground that does not contain ice wedges or lenses. When these occur, this method

may be applied only in exceptional cases where the structure is capable of withstanding considerable nonuniform settling (tens of cm).

31. An insulating layer should be placed between the permafrost and the freshly poured concrete.

B. Instructions and Considerations Regarding Stable Construction of Buildings

1. General Considerations

Regarding the aspect of stable construction on permafrost, it is necessary again to note that such construction is feasible only as a result of a complex of measures comprising the following: (1) careful investigation and survey of local conditions in accordance with a given project, (2) proper selection of the construction site, (3) proper selection of thermotechnical measures, and (4) rational choice of design and layout.

It is advisable to construct in such areas where permafrost is absent or occurs at such a depth that it does not affect the laying of foundations. Wherever the permafrost is underlain by rock, the foundations should be based on the rock even if it is frozen. If rock is not available, it is advisable to locate sites where the frozen ground is underlain by good thawed ground which can withstand high stresses. Construction on permafrost should be undertaken only when there is no alternative. Even in this case it is advisable to seek sites where the ground is sandy or gravelly and contains little moisture. It is inadvisable to erect structures on supersaturated ground consisting of fine sand, and particularly clay, loam, or silt. Such construction is permissible only when there is absolutely no other choice.

The active construction method is suitable for construction on good frozen ground containing little moisture and not subject to extensive settling when thawed. The passive construction method, involving use of all possible measures to conserve the permafrost, is suitable for construction on poor ground consisting of fine-textured material. Irrespective of the ground involved, it is essential to use designs and measures that would protect the structure against heaving of the foundation by the forces resulting from swelling of the active layer and from adfreezing between the active layer and the foundations.

2. The Passive Method of Construction

The passive method of building construction should be used only after it has been established that there is no other alternative and after adequate investigations have proved that the permafrost can be conserved near and beneath the structure. Regardless of the type and size of the structure involved, the following recommendations should be observed as much as possible and whenever feasible.

a. General Instructions

1. The natural regime of the locality should be least disturbed during both construction and consequent exploitation of the structures.

2. It is desirable to align the length of the building in the north-south direction.

3. It is advisable to shade the ground immediately adjoining the south wall of the house by means of vegetation, an awning, or a fence.

4. It is extremely important to divert the surface water, and particularly the water discharging from the roofs, from the vicinity of the buildings using the shortest route possible, so as not to permit it to penetrate into the ground.

5. Industrial water should definitely not be permitted to seep into the ground. It must be diverted most carefully and in such a manner as to avoid the use of pipes laid directly in the ground.

b. General Requirements in Construction of Buildings and Their Separate Parts

1. Reservoirs, tanks, wells, and cesspools for sewage systems and other needs, particularly for warm industrial wastes, should be located as far from the buildings as possible, and at least 15 to 20 m away.

2. Building foundations should be designed in the form of individual supports of smallest cross section possible.

3. Foundations should be sunk into the permafrost layer to a depth of 1.0 to 1.5 m, depending upon the type of structure involved.

4. Grillages consisting of two crossed layers of timbers should be placed beneath the foundations.

5. It is necessary to provide suitable air spaces beneath buildings. The minimum height of such a space is 0.5 m.

6. The air space should be carefully insulated from the interior of the building by means of a specially designed floor that would conduct from the interior an amount of heat allowed for in the design of the air space.

7. It is essential to design proper skirting that would tightly seal the air space during the warm season of the year.

8. It is advantageous to use an insulating layer of peat or slag, laid on the ground near and beneath the building, provided that this layer is protected against water. For this purpose it is desirable to locate the building above the natural ground surface, on a low fill of well-draining ground or slag.

9. Collars are permitted only in the case of unheated buildings.

c. Building Designs

As far as thermal aspects are concerned, all buildings may be divided into the following three categories: (1) unheated buildings, (2) normally heated buildings, and (3) buildings radiating large quantities of heat. In terms of material, buildings may be classified as wooden or masonry. Unheated buildings usually comprise storage buildings such as warehouses, storerooms, sheds, etc.

Crib foundations (Fig. 74) are permissible in the case of relatively small unheated buildings of secondary importance. A relatively dry site is advisable even in the case of cribwork foundations. The cribs are short logs supporting the structural frame. The upper layer of moss and vegetation is removed, and the space is filled with sand, gravel, or slag. The cribwork is laid on the fill. The cribs are placed under the corners of the building and also at intermediate points. The wooden frames should be small. The frames should be fastened together with particular care and strengthened by vertical clamps of boards or timbers. Some engineers recommend removing the natural ground beneath the cribs to a depth of 50 to 70 cm and backfilling

with slag or sand, if slag is not available, but not with either gravel or pebbles. The bottom row of the crib should be carefully and adequately tarred.

More important unheated wooden buildings should have column foundations resting on the permafrost layer. Such foundations may be of timber, masonry, or reinforced concrete. Wooden posts are best for wooden buildings. Since these foundations will be subjected primarily to heaving due to freezing of the active layer, the design of these foundations should be such as to counteract effectively the heaving. This factor should be taken into consideration when planning the design of such foundations in accordance with instructions presented later.

In the case of unheated buildings, very little change in permafrost level may be expected, particularly if the foundations consist of wooden posts of low heat conductivity. However, some change in permafrost level is probable because erection of structures and development of the locality cause the permafrost to recede. For this reason, the foundations should extend fairly deep into the permafrost. This is essential because thawing lasts three to five years after completion of the structure (Chapter II-B), while the position of the upper permafrost limit usually is determined from tests lasting only one or two years. It is obvious that such tests yield only approximate information about the position of the upper permafrost limit. On the other hand, it is irrational to practice economy in this case because lowering or raising the posts a distance of 40 to 60 cm involves a negligible part of the total cost of construction, while an insufficiently deep foundation may endanger the structure. In the case of a building measuring 12 by 12 m and involving foundation posts at intervals of 4 m, for example, lowering the posts 0.5 m involves only 6 cu m of additional excavation. Accordingly, and taking into consideration the swelling of the active layer, the posts should be extended into the permafrost a distance of 1.5 to 2.0 m, depending on the nature and thickness of the active layer.

Figure 75 shows a design of such a foundation. This design

does not permit any accurate evaluation of anchoring depth, even if a definite ultimate position of the upper permafrost limit is assumed (usually arbitrarily). The standard procedure is to regard the short anchor logs as cantilevers subjected to bending and to compute the heaving force in accordance with formula (2). Careful design of the mortises is also essential.

The foundation pit within the permafrost should be filled with the excavated material or wet sand, while the pit within the active layer should be filled with dry sand, gravel, or slag. Since the resistance to heaving is due to adfreezing between the lower part of the foundation and the corresponding ground, it is necessary to freeze the lower fill. This is accomplished by constructing the foundation in the fall or winter. The portion of the pit within the active layer should not be filled until the lower fill has frozen.

The design of masonry or reinforced concrete columns is discussed in the section dealing with construction of masonry buildings. Figure 76 shows a reinforced concrete column foundation. It consists of a simple stepped footing and a reinforced concrete column. The columns support a floor beam covered with a waterproofing layer on which the wall is raised. The footing rests on grillage consisting of two crossed rows of timbers. The ground below the beam should be removed and replaced by dry sand, gravel, or slag. A layer of clay loam or sandy loam, covered with stone paving, should be laid on top of the fill, extending to the top of the beam. The foundation pit should be filled as in the case of a wooden post. Of course, it is essential that the lower portion of the fill, in which the column is anchored, should become frozen. The foundation should extend 1.5 to 2.0 m into the permafrost.

Anchoring of foundation columns in the permafrost was recommended years ago [3, page 122]. The advantage of such anchoring has been indirectly verified at the Skovorodino experiment station. It was established that columns anchored in the permafrost to a minimum depth of 2 m did not heave under the conditions existing at Skovorodino.

Columns of rubble masonry (Fig. 77) may be used if the active layer is relatively dry and if swelling cannot occur. Such columns are

unsuitable under other conditions because they cannot withstand the tensile stresses developed due to swelling of the active layer. An exception may be made in the case when the load on the rubble foundation is sufficiently large to overcome the heaving force. It is recommended that columns of rubble masonry have tapering sides, as shown in Fig. 77, even if the ground is relatively dry.

Reinforced rubble concrete columns may prove most advantageous because the reinforcement provides the required tensile strength. The reinforcement should be placed along the face of the foundation. Masonry and rubble concrete foundations should be underlain by a grillage consisting of two crossed rows of timbers with a total height of 30 to 40 cm. The sides of such foundations should be smooth and taper at an angle of 70° to 80°.

Masonry walls erected on column foundations of rubble or rubble concrete should rest on reinforced concrete beams, while wooden walls may be erected directly on the foundations. The walls should be insulated from the foundations by four or five layers of tar paper or roof-sheeting material over cement or pitch.

The foundations of ordinary heated buildings may be similar to the previously described foundations of unheated structures, except that an air space beneath the floor as well as some supplementary design feature is an absolute requirement. The function of the air space is to prevent heat transfer into the ground and to facilitate freezing of the ground beneath the building during the winter, so as to restore the permafrost layer which thaws during the summer. The height of this ventilated air space is determined by means of appropriate calculations, but its minimum is 50 cm. The assumed height of the air space h and the thermal resistance of its ceiling R should be verified analytically on the assumption that natural circulation of the air beneath the floor is sufficient to remove the entire heat radiated by the floor of the building. In order to increase ventilation of the air space, it is advisable to construct in the middle of the building one or more vertical vents transversing the entire height of the building and having outlets in the air space. Both the vents and the air space should be tightly sealed

during the summer by means of double skirting. It is advantageous to cover the ground surface in the air space with a 20-cm layer of slag, packed moss, peat, or coniferous needles.

Figure 78 shows a rubble concrete foundation for a heated building having an air space beneath the floor. The foundation consists of a rubble concrete column extending 2 m into the permafrost and resting on a timber grillage 30 to 40 cm high. The lower section of the column, located within the permafrost, is a square prism, while the upper section, located within the active layer, is pyramidal with sloping edges. Reinforced concrete beams link the columns and support a wall three bricks thick. The beams are separated from the columns by five or six insulation layers of tar paper or Ruberoid. The ground surface in the air space beneath the floor is covered with a 15- to 20-cm layer of slag, peat, or moss. This layer is topped by a 6- to 10-cm layer of clay loam or sandy loam. The column forms have been left in the ground. The pit within the permafrost is filled with wet, fine sand, while the pit within the active layer is filled with slag, gravel, rubble, or coarse sand.

Pile foundations, sunk into the permafrost and adfrozen to it, are highly advantageous in the case of ground susceptible to considerable swelling. Figure 79 shows this type of design. The piles may be of timber, reinforced concrete, or steel. They carry a reinforced concrete floor beam which supports the wall. Pile foundations are more stable if the piles are staggered in a checkerboard arrangement, as shown in Fig. 80.

V. A. Byalinitsky designed several rational column foundations for various heated buildings. The design shown in Fig. 81 constitutes a timber foundation for a wooden building, while the design shown in Fig. 82 is a reinforced concrete foundation for a masonry building. The wooden post shown in Fig. 81 is coated with both tar paper and pitch; it seems that the tar paper is superfluous. The reinforced concrete floor beam shown in Fig. 82 is excessively shallow and should be stronger. The purpose of the board facing on the socle is to reduce the heating of the wall masonry. This procedure is less advantageous than the use of insulation along the floor beam. Such insulation is not shown in the diagram.

As stated previously, the ceiling of the ventilated air space must have definite thermal resistance because the height of this space depends on this factor. This ceiling should be properly designed, otherwise a building with a ventilated air space is unsuitable as a dwelling or workplace in the winter. According to N. I. Bikov [13, page 27], the air space cover at Skovorodino was designed as shown in Fig. 83. It consists of a subfloor made of boards 3 cm thick, an intermediate layer of slag 14 cm thick, and a finished floor made of boards 5 cm thick. In the winter the temperature at floor level was -15° C, although the room was well heated, while the temperature at table level ranged between -2° C and -4° C and the temperature near the ceiling was 40° C. Bikov approves the design of a wooden cover for a ventilated air space, shown in Fig. 84. His recommendation is made on the basis of experience at Skovorodino and Igarka. The only important comment with regard to this design has been that the wooden interflooring and beams readily tend to rot.

Figure 85 shows an optimum design of a wooden cover for an air space. The false floor consists of a double layer of boards. The lower layer, 5 cm thick, carries the load, while the upper layer, 2.5 to 3.0 cm thick, is laid across the lower layer at an angle of 45°. The boards should have groove-and-tongue joints, while the seams of both layers should be sealed with pitch. A layer of tar paper laid on cement or hot pitch should be placed between the two layers of flooring. The boards are covered with tar paper laid on a layer of pitch. This is topped by a 15-cm layer of slag. If slag is not available, use may be made of moss or shavings mixed with mortar and sand. Mortar is poured over the slag, so that the slag becomes monolithic. Lean slag concrete is the most suitable material for this purpose. At any rate, the slag is covered with a 2- to 3-cm layer of lean slag concrete. If a peat or moss fill is used, impregnated clay may be used in place of the slag concrete. The top flooring consists of painted boards 5 to 6 cm thick, well joined and carefully puttied. One of the major properties of such covers is air tightness. The design shown in Fig. 85 seems to be suitable in this respect.

Insulation or so-called greasing of the subfloor should be done with a dense material, such as slag concrete, cinder concrete, cemented shavings, or similar materials. It should not consist of loose material such as dry slag, peat waste, or shavings. Pressed boards made of reeds or straw,

fibrolite, and similar sheet materials are good insulators. At least two layers of such material should be placed over one or two layers of tar paper, and all seams should be covered. If steel beams are used in constructing the floors, the covering should be made of reinforced slag concrete whenever feasible (Fig. 86).* The slab of reinforced slag concrete is poured in wooden forms consisting of planks 2.5 to 4.0 cm thick. These forms are not removed. In American practice, the slag concrete comprises one part cement, two parts sand, and five parts slag. Reinforced slag concrete slabs are widely used in the United States and are regarded as a convenient and cheap type of construction.

Argillaceous concrete may be used in areas with deposits of clay. A 10-cm layer of light concrete, such as lean slag concrete or argillaceous concrete, is laid on top of the continuous slab of slag or argillaceous concrete. The tight wooden floor is laid over plank joists set into this layer. Hollow tiles, shown in Fig. 87, make a highly insulating and airtight cover.

Stoves should be placed on adequate insulation laid on the floor. If this is not feasible, the stoves should be placed on separate foundations. The most suitable foundation is a wooden crib filled with slag concrete, cemented wood shavings, cemented sawdust, or any other material of low heat conductivity. The bottom and top of the crib should be covered with solid rows of timbers. It is advisable to place a 20-cm layer of cement between the stove and the top of the crib. The crib is sunk to the same depth as the building foundations and passes through the entire height of the ventilated air space. If the building rests on cribwork foundations and is subject to deformation due to heaving, the stoves should not be supported on cribs because this would interfere with free deformation of the building. In this case it is advisable to use some special structure designed to transfer the weight of the stove to the walls. Figure 88 shows a design for a crib foundation beneath a stove.

In some cases it is best to erect isolated structures on fills placed directly on the ground surface. Slag is the best material for this purpose. However, well-draining materials, even stone or rubble, are suitable. An earth fill, and particularly a fill made of stone and rubble, should be laid over a layer of moss or peat and should be topped with a similar layer, with a final cover of clay or of local ground material if clay is

*On condition that the slag contain a negligible amount of sulphur, otherwise the reinforcement will rapidly deteriorate.

not available. If the fill consists of stone or rubble, it is advisable to place moss or some other good insulating material between each row of stones.

Wooden buildings on crib foundations are placed directly on the fill, as shown in Fig. 89, while masonry buildings should be erected on column foundations passing through the fill and penetrating into the permafrost to a depth of 0.75 to 1.00 m. The fill has a double function. Firstly, it prevents deep and rapid freezing of the active layer, particularly if the fill consists of material which does not conduct heat, so that the effect of swelling of this layer is lessened. Secondly, the fill prevents heat transfer to the ground and, consequently, the upper permafrost limit is conserved and may even rise. A certain house at the Skovorodino Permafrost Station has been erected in this way. It is built on cribs over a ventilated air space, but the cribs rest on the ground proper instead of on a slag cushion (Fig. 90) [13, page 28]. The slag fill or cushion beneath the building is 70 cm thick. This building was subject to heaving. This was probably due to improper construction. The cribs should have been placed on the slag fill, that is, they should have been raised 70 cm. The fill near and beneath the building should have been covered with a layer of earth over moss, so that the earth material would not penetrate into the slag. In addition, this layer of earth material would have prevented free air circulation within the fill, so that the insulating effect of the fill would have improved. Finally, a layer of moss should have been placed in the fill. At any rate, the permafrost beneath this building was conserved and the deformation resulting from swelling of the active layer was slight. The house continued to serve as a dwelling after the flooring had been repaired so as not to admit excessive cold (Fig. 83). A number of houses, located in other places, were placed on an 80-cm layer of slag, as shown in Fig. 89.

At the Urusha Station, fills of this kind were made of boulders. Available information indicates that the permafrost beneath this building has thawed, but this apparently was due to improper design of the fills, since the fills had inadequate insulation layers at the top and bottom and had no layers of moss or other insulating material between the individual rows of boulders. In addition, it is unknown whether the ventilated air spaces beneath these buildings were properly designed. If the bouldery fills had been covered with moss and local earth, and if it had been packed with moss, straw, or similar material, the permafrost would not have thawed.

Buildings radiating much heat may be designed as described above. However, since some of these buildings may require extremely high air spaces and even special ventilation equipment, it is advisable to build them two stories high. In this way, the lower floor could be maintained at ordinary temperature, while the heat sources could be located on the second floor. Such an arrangement is particularly advantageous in the case of wooden buildings such as bath houses, laundries, large kitchens, etc.

The discharge of water, steam, and gases from industrial installations located in such buildings should be arranged with great care, using insulated pipes located outside the building in such a way that they would not transfer heat directly to the walls, foundations, and ground, and the discharge water should be prevented from seeping into the ground near the building. In addition, it is essential that the walls and foundations are not heated directly by the apparatus and installations. In the well-known case of a building in which two forges were placed near a wall, the wall became intensely heated, with the result that the permafrost beneath the building thawed.

It is feasible and advantageous to provide ventilated air spaces for heated buildings consisting of one or more floors if these buildings are of the common type and do not have exceptionally wide bays. In the case of workshops or other heated buildings having large bays, ventilated air spaces can be constructed by placing the flooring on piles (Fig. 91) sunk into the permafrost by means of an American steam point. In practice, however, the size of the building may necessitate an excessively high air space. This is not always feasible and advantageous.

In the case of factory buildings, the floor above the air space might be both heavy and expensive even if the building is small. Many industrial buildings house heavy equipment, while some are designed to accommodate trains. Regardless of the temporary loads, the flooring over a ventilated air space would be quite heavy because of the layers of insulating material. Therefore, often it is not feasible to construct a ventilated air space, and it is necessary to lay the floor directly on the ground. If such buildings are erected on permafrost, the only solution is artificial conservation of the permafrost by means of special refrigeration installations or by

excavating a deep pit and filling it with suitable material. Although such procedures seem logical and feasible, they will not be discussed here in detail because they have not been adequately worked out as yet.

It is obvious that the principle of permafrost conservation is unsuitable as yet for construction of buildings radiating much heat. Such buildings should be erected on sites where permafrost is absent or bedrock occurs at shallow depth. In addition, other construction methods may be used.

The ventilated air space can be designed in accordance with the formulas and recommendations prepared by Tsytovich [2, page 400]. It involves determining the thermal resistance of the floor R, which is the reciprocal of the universal coefficient of heat transfer K used in construction design. The value of R for an air space cover of a given design is determined from the formula

$$R = \frac{1}{K} = \frac{1}{a} + \frac{1}{a_o} + \sum_1^n \frac{l_i}{\lambda_i}$$

in which a and a_o are the total magnitudes of the convection and radiation coefficients of the inner and outer surfaces of the building floor, l_i is the thickness of each of the layers of material comprising the floor, and λ_i are the coefficients of heat conductivity of each of these layers. The algebraic sign of the sum $\sum_1^n$ indicated that it is necessary to use the sum of the ratios $\frac{l_i}{\lambda_i}$ for all the layers of material comprising the floor. The values of a, a_o, l_i, and λ_i can be obtained from handbooks of heating and ventilation.

The allowable quantity of heat radiating from the building floor into the open air space should be

$$q_o = Q_1 - \frac{h_2}{h_1} Q_2$$

in which

Q_1 is the quantity of heat lost annually by the ground during cooling of the uncovered area,

Q_2 is the quantity of heat gained annually by the ground during warming of the uncovered area,

h_2 is the allowable depth of thawing of the ground in the air space, and

h_1 is the depth of thawing of the uncovered ground.

The thermal resistance of the floor R, determined from the formula, should be not less than the value corresponding to the allowable quantity of heat radiating from the floor. That is,

$$R \geq \frac{(T - t_c)\ 24\ m}{q_o}$$

in which

T is the temperature within the building,

t_c is the mean temperature in the air space during the period of negative temperatures, and may be taken as the mean temperature of the outside air during the same period, and

m is the number of days in the year during which the temperature is negative.

The values of Q_1 and Q_2 are determined from the formula

$$Q = h_1(c_1 w\delta t_1 + c_2\delta t_1, + 80\delta w + w\delta t_2 + c_2\delta t_2)$$

in which

h_1 is the height of the ground layer involved, in centimeters,

$c_1 = 0.5$ is the specific heat of ice,

c_2 is the specific heat of dry ground,

δ is the volume weight of the oven-dry ground,

w is the moisture content of the ground by weight, expressed as a fraction,

t_1 is the negative temperature at the beginning of the time interval under consideration, and

t_2 is the positive temperature at the end of this time interval.

The approximate depth of thawing in the air space can be determined from the following equation:

$$h_2 = h\,\frac{0.85\ Q_2 - q_2}{Q_2}$$

in which q_2 is the quantity of heat radiated by the building floor during the summer, and h is the depth of the active layer. The depth of freezing of the ground in the air space is

$$h_1 = h \frac{Q_1 - q_1}{Q_2}$$

in which q_1 is the quantity of heat radiated by the building floor during the winter.

The required height of the air space is largely determined in accordance with the following analysis by Tsytovich. The air space will be ventilated, that is, adequate air exchange will occur if the available air pressure H is equal to or greater than the pressure loss in the openings H_1. Accordingly,

$$H \geq H_1$$

The value of H can be determined if it is assumed that the neutral zone occurs at the middle of the height l of the ventilated air space (Fig. 92).

$$H = 0.5\, l\, (\gamma_{tH} - \gamma_{tc})$$

in which γ_{tH} is the weight of the air at the given temperature of the outside air, and γ_{tc} is the weight of the air at the mean temperature of the air in the air space. The weight of the air is determined from the formula

$$\gamma_t = \frac{1.293}{1 + \frac{t}{273}}$$

The magnitude H_1 is

$$H_1 = \frac{v^2 \gamma_{tc}}{2\, g a^2}$$

in which

v is the air velocity in the plane of the opening,

γ_{tc} is the specific gravity of the displaced air,

g is the acceleration of gravity, and

a is the contraction coefficient, usually taken as 0.65.

In this formula, v is in meters per second, γ_{tc} is in kilograms per cubic meter, g is in meters per square second. Accordingly, H is in kilograms per square meter or in millimeters of water. The minimum height of the air space, which is the distance between the ground surface and the bottom of the floor beam, should be not less than 50 cm.

The recommended procedures for reducing the effect of the active layer are presented in B-5 of Chapter IV.

The OST Manual No. 90032-39 contains the following special instructions regarding the design of foundations under the conditions discussed above.

OST No. 90032-39

32. As a rule, foundations should extend below the active layer. In the case of unimportant and temporary buildings, it is permissible to use simplified foundations such as cribs, ground plates, blocks, etc., which are placed on the ground after the surface layer of the ground has been removed. In the case of important buildings, foundation bases in the active layer are permissible only if this layer consists of dry sand and gravel, if it is feasible to keep the ground dry, and if the underlying strata are suitable.

33. When construction method A (the passive method) is used, the depth of the foundations is determined in accordance with the conditions necessary to conserve the permafrost at the base, with due consideration of the thermal regime of the structure, the temperature regimes of the active layer and permafrost, as well as the design measures to prevent thawing. In the case of important buildings, the stability of the permafrost at the base should be verified analytically. Foundations constructed on merging permafrost by either the passive or active method require consideration of the fact that the thickness of the active layer may increase, particularly near the south walls.

It should be noted that item 32 above contains some erroneous instructions regarding the feasibility of basing foundations of important buildings on the active layer. Those instructions and recommendations should have been omitted.

d. Notes on Foundation Calculations

The allowable stresses in permafrost must be determined when

designing foundations based on the permafrost layer. The allowable load on the permafrost at a given construction site can best be determined in the field, in accordance with the instructions presented in Chapter III, B-3b. OST Manual No. 90032-39 contains a special table of allowable loads on permafrost and recommends the following special procedures for determining these loads.

OST No. 90032-39

17. Allowable loads on permafrost are determined from results of adequate geological, engineering, and permafrost investigations of the ground beneath the projected structure. Consideration must be given to the foundation design for individual structures, their temperature regime, and the rigidity of the structure.

18. In the case of structures erected on permafrost having constant negative temperature and pores completely saturated with ice, the allowable loads may be taken from OST Table II.

19. In cases where the permafrost is likely to thaw (beneath heating plants, for example, or if the permafrost temperature is zero), the allowable load should be determined from results of load tests conducted on the thawed permafrost.

20. The bearing capacity of "dry permafrost" is determined from investigations conducted in the same way as in the case of ordinary thawed ground. The bearing capacity of "dry permafrost" is the same at temperatures below and above zero.

3. Building Construction Designed To Allow for Settling Due to Thawing of the Permafrost

This construction method is not recommended for relatively important structures if the ground is wet and the permafrost contains poor material. This method is quite suitable in the case of sand, gravel, and rubble with a maximum moisture content of 25 to 30 per cent and containing no ice wedges or lenses. Under these conditions, it is unnecessary to take extreme measures to conserve the permafrost because the low moisture content and good engineering properties of the ground will prevent considerable and hazardous settling of the foundation during partial thawing of the permafrost beneath the foundation. Accordingly, the foundation should be designed on the assumption that the permafrost will thaw, and the allowable load should be taken as that for thawed ground, determined by field tests. The foundation should extend through the upper wet layer of the permafrost

OST Table II

ALLOWABLE COMPRESSIVE STRESSES FOR ICE-SATURATED FROZEN GROUND

No.	Classification by Texture	Allowable Stresses in kg per sq cm at Temperatures		
		-0.2 to -0.5 C	-0.5 to -1.5 C	-1.5 to -2.5 C
1	Sands (Fractions: <1 mm, 100%; < 0.005 mm, 3% or less)	3.5	4.5	6.0
2	Loamy Sands (Fractions: < 0.005 mm, 10% or less)	2.5	3.5	4.5
3	Sandy Loams (Fractions: < 0.005 mm, 10 to 30%)	2.0	3.0	4.0
4	Clay Loams (Fractions: < 0.005 mm, in excess of 30%)	1.5	2.5	3.5
5	Silty Clay Loams (Fractions: ~~0.01 to 0.005~~ mm, in excess of 50%; < 0.005 mm, up to 30%; organic matter up to 10% in some cases)	1.0	2.0	3.0

Remarks:

a. The allowable stresses at intermediate temperatures are determined by interpolation.

b. The values are not applicable to permafrost containing ice inclusions.

c. The values may be increased 1.5 times when applied to third class structures.

(Fig. 3), the distance involved being not less than 1 m.

The extent of probable settling should be determined at the construction site in accordance with instructions given in Chapter III, as well as analytically. Intensive settling can be prevented by retarding the thawing of the permafrost. This can be accomplished to some extent by using the procedures presented in B-2 of this chapter. Other measures include the design shown in Fig. 91, as well as placing beneath the building floor a cushion of slag or cinders, 30 to 50 cm thick and extending 1.5 to 2.0 m beyond the building (Fig. 93). Nonuniform settling and its effects can be reduced by using also the procedures recommended in items 28 and 29 of the OST Manual No. 90032-39.

In the case of buildings where it is feasible and relatively cheap to construct ventilated air spaces, they may be constructed as described previously, provided the flooring above the spaces has high thermal resistance. A ventilated air space is particularly recommended in the case of ordinary structures erected in areas where the permafrost temperature is near zero and not lower than -0.5° C, since the probability of thawing or conservation of the permafrost under such conditions is rather indefinite, so that it is safer to assume that it will thaw, yet it is essential to take measures to retard or even stop this process which may produce highly undesirable effects. Wooden buildings of secondary importance may be erected on cribs or grillages. The deformation of a house on cribs, caused by thawing of the permafrost, can be rectified. Such buildings may be erected even on ground containing more than 30 per cent moisture. The design of wooden houses on cribs does not differ from the designs described previously.

Pile foundations recently gained wide usage in industrial construction. They are built as follows. The piles are extended into the permafrost to a depth of 5 to 6 m on the assumption that they carry only part of the load, while the rest of the load is transmitted through the foundation platform to the ground compressed solidly in the process of pile driving. This assumption is doubtful, since the piles will settle if the ground is weak and excessively moist. This design is satisfactory in the case of good ground of low moisture content, but piles are hardly necessary under such conditions. If deformation has not occurred as yet in structures erected on

such foundations, it is probably due to the fact that the permafrost along and beneath the piles has not yet thawed.

A continuous mat beneath the entire structure is a suitable foundation in the case of poor permafrost. Such a reinforced concrete slab, properly designed and being exceedingly rigid, prevents deformation of individual parts of the structure but permits uniform deformation of the entire structure, which occurs in the form of slight tilting when thawing is greater on one side. Foundations of this type are widely used in permafrost regions for water towers and pumping stations occupying relatively small areas. The reinforced concrete slab in these cases is about 1 m thick and is laid on a fill of sand or gravel, or on a foundation platform consisting of two crossed rows of timbers.

Whereas water towers on ordinary ring foundations developed large cracks, those on flat slabs remained in satisfactory condition. A leaning water tower, erected on a flat slab, was found at a certain railroad station. Despite the fact that it leaned, there were no cracks in the walls. The leaning resulted from uneven thawing of the permafrost on the side where the water mains passed through the ground. Engineer Budayev describes a wooden bell tower at the same station. On November 28 this tower leaned southward 28 cm with respect to the vertical, while the following April 18 it leaned only 14 cm. This discrepancy was apparently due to swelling of the ground. By May 10 the leaning amounted to 16 cm. While this tower did not rest on a flat slab, its construction was very rigid, its dimensions were small, and it rested on a timber platform so that this tower may be compared to some extent with a building founded on a continuous slab.

Figure 94 shows M. Ya. Chernyshev's foundation design for masonry water towers on supersaturated loam and silt which turn into slud upon thawing. The foundation consists of a rubble wall resting on a continuous concrete slab laid on a platform consisting of two crossed layers of timbers. The outside of the foundation is carefully smoothened to avoid the harmful effects of swelling of the active layer, and the surrounding space is backfilled with gravel, rubble, or slag.

It is timely at this point to call attention to an interesting instance of construction on very poor ground. An eleven-story building was

erected in Albany, New York, on clay loam containing about 50 per cent water. The foundation of this building consisted of a continuous reinforced concrete slab 52 m long, 32 m wide, and 0.91 m thick, on which were arranged longitudinal and transverse reinforced concrete Vierendeel beams spaced 6 m apart and supporting the columns [17]. Of course, the ground involved was not permafrost, yet it was very poor ground. Hence, if a foundation of this type proved rational for a heavy building under these conditions, it seems logical to assume that similar foundations would be suitable in some cases under permafrost conditions.

When construction is based on the assumption that the permafrost will thaw and the foundations will settle, it is necessary to know the order of magnitude of the settling, the intensity and depth of settling during a given period, as well as the allowable load on the thawed ground. These data are obtained from tests and investigations described in Chapter III-B, as well as from the following analyses. It should be noted that the magnitude of settling of thawed ground under load depends largely on the depth of thawing. This depth is an extremely complicated function of various fairly indeterminate factors. Accordingly, Tsytovich and Sumgin are of the opinion that it is impossible accurately to determine the magnitude of settling of foundations on thawing permafrost. Even an approximate estimate of settling is possible only if the weight of the structure does not exceed the critical load for the given type of ground when thawed. If the load is supercritical, the resultant settling might become excessively large because the ground beneath the foundation may be forced out. This should not be permitted to occur because it would inevitably result in complete failure of the structure. This phenomenon nearly always occurs in the case of clay loam because it usually has practically no strength when thawed. The intensity and approximate depth of thawing can be determined from the formulas and analytical considerations presented previously.

The depth of thawing may be calculated from M. M. Krylov's formula

$$h_m = h'_m + \sqrt{\frac{2\theta\lambda t}{w\rho}} - \frac{qt}{w\rho}$$

in which

h'_m is the depth of thawing at the beginning of the period involved, in meters,

h_m is the depth of thawing at the end of the period involved, in meters,

θ is the drop in temperature,

λ is the coefficient of internal heat conductivity of thawed ground,

t is the period of thawing, in hours,

w is the moisture by weight relative to dry ground,

ρ is the latent heat of fusion of ice (80,000 cal per cu m), and

g is the heat lost to the permafrost.

The magnitude of the critical and allowable load on thawed ground should be determined from proper tests at the construction site. An approximate value of the critical load can be obtained from the following formula developed by Prof. N. P. Puzirevsky (Frelikh) [18, page 235]:

$$\sigma_{cr} = \frac{\pi \gamma H}{ctg\phi - (\frac{\pi}{2} - \phi)}$$

If it is not feasible to determine the allowable load on the permafrost by means of field tests, the following formula may be used in the preliminary design:

$$\sigma_{cr} = \frac{0.67 \pi \gamma H}{ctg\phi - (\frac{\pi}{2} - \phi)}$$

In these two formulas, γ is the volume weight of the ground in tons per cubic meter, H is the foundation depth in meters, and ϕ is the angle of internal friction of the ground. Neither formula is applicable to clay.

The method of equivalent ground layer can be used to calculate the approximate value of ultimate settling in the case when the critical load on the given ground is greater than the load due to the structure. This method takes into account the entire depth of the compressed ground layer beneath the foundation, the physical properties of the ground, and the dimensions and

shape of the foundation. This method is not applicable to highly heterogeneous frozen ground containing ice wedges. The complete and ultimate settling of the foundation would be [2, page 410]

$$S = h_s \frac{\epsilon_1 - \epsilon_2}{1 + \epsilon_1}$$

in which

- h_s is the thickness of the equivalent ground layer,
- ϵ_1 is the initial porosity coefficient of the undisturbed ground, which is the porosity coefficient of the frozen ground the volume of which includes vapor and ice, and
- ϵ_2 is the porosity coefficient of the stressed ground after settling of the thawed ground has ceased.

According to Tsytovich,

$$h_s = Aw_o b$$

in which b is the width of the foundation base (of length a), and the value of the product Aw_o is taken from Table VII compiled by Tsytovich.

4. Building Construction Utilizing Preliminary Elimination of the Permafrost at the Base and Methods Used When the Permafrost Occurs at Considerable Depth

The method of preconstruction elimination of the permafrost is quite suitable in cases where the permafrost layer is thin and where the permafrost is layered or spo dic. However, this method may also be used when the permafrost is thick but has a relatively high temperature and when the permafrost contains little moisture, even if it is located at a depth of 5 to 6 m below the ground surface. As stated previously, the moisture content of the permafrost layer rapidly decreases with increasing distance from the upper permafrost limit (Fig. 3). It is possible, therefore, that the moisture in the permafrost at a depth of 5 to 6 m would be so small that the underlying layers would hardly settle during thawing and this settling would not affect the structure appreciably. The wet upper layers would settle during preliminary thawing and no additional settling will occur. Accordingly,

TABLE VII

VALUES OF Aw_o FOR COMPUTING THE EQUIVALENT GROUND LAYER IN DETERMINING THE AVERAGE SETTLING OF THE ENTIRE AREA UNDER LOAD

Shape of Area Under Load	$\frac{a}{b}$	Poisson's Coefficient μ							
		0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45
Circle	-	0.97	0.99	1.02	1.08	1.18	1.34	1.73	2.81
Square	1	0.96	0.97	1.01	1.07	1.17	1.32	1.71	2.78
Rectangle	1.5	1.16	1.18	1.23	1.30	1.40	1.60	2.07	3.37
"	2	1.31	1.34	1.39	1.47	1.60	1.81	2.34	3.81
"	3	1.55	1.58	1.63	1.73	1.89	2.13	2.75	4.48
"	4	1.72	1.75	1.81	1.92	2.09	2.36	3.06	4.98
"	5	1.85	1.88	1.95	2.07	2.25	2.54	3.29	5.36
"	6	1.98	2.02	2.09	2.21	2.41	2.72	3.53	5.64
"	7	2.06	2.10	2.18	2.31	2.51	2.84	3.67	5.98
"	8	2.14	2.18	2.25	2.40	2.61	2.95	3.82	6.21
"	9	2.21	2.26	2.34	2.47	2.69	3.04	3.92	6.42
"	10	2.27	2.32	2.40	2.54	2.77	3.13	4.05	6.59
"	20	2.67	2.75	2.82	2.98	3.25	3.67	4.75	7.73
"	30	2.91	2.97	3.08	3.25	3.54	4.00	5.18	8.44
"	40	3.10	3.16	3.28	3.47	3.77	4.27	5.53	8.99
"	50	3.25	3.32	3.44	3.64	3.96	4.47	5.80	9.43
"	100	3.73	3.80	3.94	4.16	4.54	5.12	6.64	10.81

normal conditions of construction on thawed ground will prevail.

Thawing of the ground may be partially accomplished by utilizing natural heat. This requires large scale melioration of the locality early in the spring. Melioration comprises removing the scrub and other minor vegetation, removing the moss or peat cover, removing the layer of humus, installing a system of drainage ditches, and burning the grass cover. Building fires on the ground surface results in deeper thawing of the permafrost.

Steaming the ground by means of an American steam point is the best method for thawing the permafrost. Such steaming to a depth of 5 to 7 m presents practically no difficulties. Since the ground usually is quite wet after the steaming operations, it is necessary to remove the water from the ground by means of proper drainage or even by pumping if drainage is not feasible. Pumping is a speedier method. Such pumping may utilize the apparatus and installations which are widely used in construction to lower the level of the ground water [15, page 235]. Experience in lowering the level of ground water proves that the method is effective if the depth does not exceed 20 to 25 m and if suitable pumps are used. Of course, the effectiveness of this method depends primarily on the nature of the ground involved.

The plan of a certain dam on the Volga River, where the ground is sandy and silty, requires lowering the level of the ground water 26 m below high level. The maximum lowering achieved abroad is 24 m.

Figure 95 is an illustrative example. It is a schematic diagram of the latest American installation, the Moretrench Wellpoint [19], used under ordinary conditions on construction of a sewage system in North Toronto, Canada. The ground consisted of layers of sandy loam, silt, clay, and clay containing gravel and stones. Within 49 hr after the start of pumping, the level of the ground water was lowered to the required position, that is, 6 m. The entire installation consists of a main laid around the excavation, special wellpoints arranged at intervals of a few meters (Fig. 96), and several pumps of high capacity. The well consists of a bore into which a steel pipe is lowered. The pipe is protected by a wooden jacket. The diameter of the bore is somewhat larger than the diameter of the jacket. The clearance between the bore and the jacket is filled with clean sand which acts as a filter. Figure 97 is a general view of the excavation and the pumping

system. A steam shovel is operating in the excavation. The work proceeds as on a dry area.

Settling ceases after the water is removed from the ground. It is feasible then to grade the area and to proceed with construction operations in the usual manner. After the water has been removed, weak ground can be strengthened by piles or other means.

The question arises as to whether permafrost might recur and result in possible deformation of the structure due to swelling of the newly frozen ground which, in addition, has become wet due to seepage from the adjoining strata. With reference to this, it should be noted that relatively little water will seep into the ground because the ground has already been compressed. Refreezing of the ground will proceed very slowly, in very thin layers each year. The temperature of the freezing layer cannot be very low under these conditions. Therefore, deformation of the erected structures would be correspondingly inappreciable. It should be noted that no actual cases are known where a rise in permafrost level has had any negative effect on the structure. No deformation is known to have occurred due to such a rise. On the contrary, a rise in permafrost level is desirable because it would increase the strength of the foundation base. In the case of structures erected by means of the method under consideration, however, recurrence of the permafrost is improbable because the buildings involved are heated buildings with floors resting directly on the ground or buildings radiating large quantities of heat.

In the case of buildings erected by this method, septic tanks, wells, mains, and piping for sewage and industrial waste may be located within the building or in its immediate vicinity. All the melioration measures should be continued after the structures are in use and new measures should be undertaken. It is recommended not to remove the snow from the immediate vicinity of the building during the winter. It is essential to remove as little as possible and to practice artificial conservation of the snow near the building.

The depth of foundations should exceed the depth of winter freezing of the ground. The latter depth should be determined on an area from which

the vegetation cover has been removed. It is essential to install a drainage system around the building. This system comprises a wooden pipe laid about 1 m below the active layer and covered with rubble or gravel, and an open ditch 0.60 to 1.00 m deep and 0.30 to 0.40 m wide at the bottom. Figure 98 shows the arrangement and location of this system with respect to the building. The function of the drain is to decrease the moisture in the ground in order to prevent excessive swelling of the active layer. Other measures of similar nature are presented in B-5 of this chapter.

With regard to construction of buildings in areas where permafrost is absent or occurs at great depth, the design of wooden buildings differs from that of masonry buildings. Both designs depend on the composition and moisture content of the active layer because deformation of such buildings can result only from swelling of the active layer. When the active layer swells and is excessively wet, wooden buildings usually are designed in the form of frame structures capable of resisting deformation. Such resistance can be achieved by making extremely rigid corners, using vertical clamps fixed with bolts, doweling of individual timbers, and securely bolting the lower timbers.

As in the previous instances, wooden buildings of secondary importance may be erected on cribs. The air space beneath the floor should be insulated, so that the ground would not be subjected to intensive freezing during the winter and excessive swelling would be avoided. Therefore, a plank skirting is built around the outside base of the building (Fig. 99) and is filled with an insulating mixture of sand with peat, moss, sawdust, coniferous needles, etc. The flooring may consist of a false and regular floor, but it should be built so as to assure maximum air tightness and insulation and satisfy the standard requirements of an adequate temperature at floor level. The most suitable pugging material for the false floor is a relatively dense material such as slag concrete, cemented sawdust, and similar materials, but not loose materials such as plain slag, peat waste, or sawdust.

A ditch, 0.60 to 1.00 m deep and 0.30 to 0.40 m wide at the bottom, as shown in Fig. 98, should be dug at a distance of 3 to 5 m from the walls in order to reduce the moisture in the active layer around the

building. If the building is large, the foundations may consist of individual concrete columns extended at least 0.5 m below the active layer. For this purpose, the depth of winter freezing should be determined at the construction site on an area from which the surface cover has been removed. The sides of the columns should taper at an angle of 70° to 80° and should be smoothened with cement mortar. The tensile strength and resistance to heaving of these concrete columns should be tested in accordance with the instructions presented in Chapter II.

If the ground is suitable for driving piles, the concrete columns may be replaced by timber piles. The piles should extend at least 1 m below the active layer. Reinforced concrete piles may be used in the case of major and heavy structures. A fill of nonswelling material should be placed around the piles and columns within the active layer. The fill should have a radius of 0.75 m and may consist of rubble, gravel, or screened coarse slag, soaked in fuel oil, naphtha, or tar, and protected against silting. It is advisable to provide an insulated air space beneath the floor and to drain the ground even when footings or pile foundations are used, particularly if the active layer consists of material which swells readily.

The foundations for masonry structures on an active layer consisting of well-draining material may be selected arbitrarily, provided the design allows for adequate drainage around the building and a warm air space beneath the floor, which is closed off during the winter. If the active layer consists of wet and readily swelling material (fine sand, silt loam, and similar fine-textured materials), the building foundations should be designed in the form of individual columns of masonry or reinforced concrete. Columns of rubble or rubble concrete may be used only when the load on them is sufficient to prevent heaving. This factor can be established by means of computations based on the instructions presented in Chapter II.

Masonry columns should have a smooth finish of cement mortar, and their sides should taper at an angle of 70° to 80°. Reinforced concrete columns are designed in the form of relatively narrow pillars with vertical sides, resting on relatively broad footings, as shown in Fig. 76, but without the grillage platform beneath the footing. As stated previously, the natural ground around the columns should be removed and replaced with material that

does not swell and does not readily adfreeze to the foundations.

Masonry buildings more than two stories high may be erected on continuous rubble foundations if their air spaces or basements are insulated, if drainage is installed around the buildings, and if the foundation pits are backfilled with rubble, gravel, or slag. Industrial buildings (shops, factory plants, depots) should be erected on separate foundations of concrete, rubble, or piles, if the active layer is subject to heaving. The piles or other supports should be designed to resist tension and heaving. If the piles must pass through the active layer, they should be driven to a depth equal to twice the depth of the active layer plus 1 m. In the case of foundations extended below the active layer, the piles should be driven to the required depth and should be tested for resistance to heaving (Chapter II, B-1), the magnitude of which is determined in accordance with local conditions.

Water tanks and other small structures may be erected on continuous foundations consisting of reinforced concrete slabs. Concrete, rubble concrete, or reinforced concrete foundations, constructed in an open pit or by means of sunk wells and caissons, are recommended for cases of deep foundations such as those described in Chapter IV, B-2, as well as in the case when permafrost is absent but the ground near the surface is weak and the active layer is subject to considerable swelling.

A foundation column extended to bedrock may be constructed in an open pit by means of sheet piling. The sheet piling is placed in two stages. The upper row is made considerably wider than the foundation, so that the lower row could be installed after the upper part of the pit has been excavated. The sheet piling is braced, and the lower row serves as the form for pouring the concrete. The sheet piling may be wood or steel. Figure 100 shows a 23-m foundation column constructed in the United States in an open pit under ordinary conditions. The sheet piling is steel. The dimensions and design are shown in the drawing. The bracing consists of I-beams set into horizontal I-beam belts. Steel sheet piling is appropriate under our conditions and is recommended for this type of operation, particularly since engineers in the USSR have adequate experience in this method of construction. Steel sheet piling is manufactured now at rolling mills in the USSR.

Thawing of the permafrost in the foundation pit is readily accomplished by means of steam points. In individual cases where circumstances are favorable, thawing of the permafrost may be achieved by means of electrical heating. This method was used on construction at Solikamsk [15, page 288] and is described in Chapter V.

Sunk wells and caissons may be of wood, metal, reinforced concrete, or even ordinary concrete. Since the supports often must resist tensile stresses caused by swelling of the active layer, it is advisable that caissons be of combined construction: a working chamber of wood, and the remainder of concrete reinforced with round steel bars which furnish the tensile strength (Fig. 101). Masonry is unsuitable because its tensile strength is low. Sunk wells in permafrost should be made of wood, in the form of horizontal timbers with an outside bracing of planks 5 to 7.5 cm thick (Fig. 102), depending upon the dimensions of the well. The planks should be fastened to each timber or to every second timber by means of at least two nails apiece. The timber beams should be bolted together. A similar design may be used in the case of caissons, and the space between the wooden wall and the shaft may be filled with masonry.

Wooden caissons and sunk wells should have a rectangular horizontal section. A circular shape is preferable in the case of caissons and sunk wells made of concrete or reinforced concrete because this shape facilitates construction operations and yields the largest relative bearing area. The diameter of the foundation column in a sunk well or caisson should be determined from calculations, but it should be not less than 90 or 100 cm in order to facilitate excavation. The working chamber of a caisson usually is about 2 m high; if the column diameter is small, the chamber should be designed to accommodate a single workman. Sunk wells may have removable metal casings which are gradually removed as the column is being constructed [15, page 288]. Sunk wells and caissons are advantageous because they enable construction of deep foundations even under the most unfavorable conditions. The diagrams presented previously are based on construction experience under ordinary, nonpermafrost conditions. However, similar designs may readily be utilized under permafrost conditions. In the case of sunk wells and caissons in permafrost, the

pits may be prepared by thawing the ground with steam points, electrical heaters, or hydraulic dredging with hot water. The tensile stresses due to swelling and the corresponding reinforcement may be computed in accordance with instructions presented in Chapter II, B-1.

5. Measures Against Heaving of Foundations

Swelling of the active layer is inevitable, regardless of the construction method used. Therefore, it is necessary to apply a series of measures which would protect the building foundation against heaving. Essentially, these measures are as follows.

Foundations should be built in the form of individual supports of the smallest section possible, designed to become wider with increasing depth in the active layer. Their sides should taper at an angle of 70° to 80° with respect to the horizontal and should be smoothly finished with cement mortar. Wooden columns should be smoothly planed and tarred. It is advantageous to fill the pit around the foundation with gravel. Prior to filling, the gravel should be mixed with naphtha, fuel oil, or coar tar. Planks or a sand filter should be used to protect the fill against silting. The ground surface around the building should be covered with insulating material in order to reduce the intensity of freezing. If the permafrost occurs at great depth, it is essential to provide an insulated air space beneath the floor.

If the active construction method is used, it is essential to dry the ground around the building as much as possible. If either of the other two construction methods is used, drainage is permissible only to the extent that it does not interfere with conservation of the permafrost or with retardation of its thawing. Foundations must be sufficiently strong in tension to resist the effect of swelling of the active layer.

Piles within the active layer must be smoothly planed and coated with tar, pitch, fuel oil, or crude oil. Prior to coating, all large cracks in the pile should be filled with a suitable material. The active layer around the pile should be excavated within a radius of 1.0 to 1.5 m and the pit should be filled with gravel of 5 to 10 cm in diameter and with a layer

of sand, placed within a wooden frame without a bottom. The fill should be placed in successive layers of thickness equal to the width of the frame plank. A 25- to 30-cm layer of peat or moss should be placed on the bottom of the pit prior to filling. The fill should extend to within 0.6 m below the ground surface and should be covered with 20 to 30 cm of moss, peat, or straw. The remaining part of the pit should be filled with well-tamped local earth. Figure 103 is a diagram of this type of fill.

The OST Manual No. 90032-39 gives the following detailed instructions regarding the aspect under consideration:

OST No. 90032-39

34. Particular attention should be given to protecting the foundations and the entire structure against deformation due to heaving forces and icing phenomena. Accordingly, the following measures are recommended:

(a) To dry the area by means of grading; to build an impervious barrier around the structure; to install troughs and paved ditches for rapid removal of atmospheric precipitation; to provide drainage installations that would not freeze, etc.

(b) To place insulation layers on the ground near the foundations so as to reduce the thickness of the active layer.

(c) To eliminate or reduce the effects of hydrostatic and hydrodynamic pressures by using deep drainage, frost belts, and frost dikes.

35. The following measures are recommended to reduce the adfreezing between the active layer and the foundation:

(a) To fill the pit around the foundation with coarse and nonswelling material such as gravel, rubble, or coarse sand. The backfill should be well drained and protected against silting.

(b) To taper (not more than 70° with respect to the horizontal) the sides of the foundation columns and to smoothen their surfaces. Foundation surfaces should be smoothened or iron plated, while timber piles and posts should be planed.

36. Structural foundations should be designed to resist ejection from the ground and failure due to the adfreezing forces. The allowable shearing stresses and the adfreezing stresses involved are presented in OST Tables III and IV.

37. Heaving of foundations due to adfreezing forces can be counteracted as follows:

(a) To reduce the number of foundation columns by increasing the load on each column and to decrease the cross section of the column within the active layer so as to decrease the adfreezing surface.

OST TABLE III

ALLOWABLE SHEARING STRESSES FOR FROZEN FINE-TEXTURED GROUND (LOAMY SAND, CLAY LOAM AND SILTY CLAY LOAM)

Ground Temperature in Degrees C	Allowable Stress in Kg per Sq Cm
0	0.5
-0.5	1.5
-1.0	2.5
-1.5	3.5
-2.0	5.0

(b) To anchor the foundations in the ground below the active layer, extending them the required distance into the permafrost layer or the thawed layer.

(c) To design the foundations in such a way that they would withstand the tensile stresses due to heaving.

38. The following measures can prevent deformation due to possible horizontal displacement of structures located on ground which swells readily and has a high water table, particularly if basements are included in the plan:

(a) To reduce the distance between cross walls, and to construct buttresses, cross foundations, or rigid beams in the space between these walls, if necessary.

(b) To strengthen the foundations by constructing rigid reinforced concrete belts.

C. Instructions and Considerations Regarding Stable Design of the Road Structures.

1. General Remarks.

The essential aspect of design and construction of road structures is that stability can be achieved only be means of a series of various measures comprising proper selection of site, design, material, and insulation, as well as proper engineering procedures.

Proper selection of a site is a considerable task. It involves consideration of the entire complex of topographic, geological, hydrogeological, permafrost, and ground conditions. It is necessary to endeavor to locate these structures in areas where bedrock occurs at or near the ground surface, in areas where the active layer consists of coarse material or solid clay, where the upper permafrost limit occurs at great depth, or where

OST TABLE IV

STRESSES DUE TO ADFREEZING BETWEEN GROUND AND WOOD OR CONCRETE IN KILOGRAMS PER SQUARE CENTIMETER

Adfreezing Surfaces	Temperature - 1° C, Ice Saturation				Temperature - 10° C, Ice Saturation			
	0.25	0.50	0.75	1 to 1.4	0.25	0.50	0.75	1 to 1.4
Fine-textured ground (loamy sand, sandy loam, clay loam, silt) and wood	2	3	4	6	3	7	13	16
The same ground and concrete	1	2	4	5	7	10	13	16

General Remarks:

1. The stresses at other temperatures and ice saturation are determined by interpolation.

2. In the case of gravel protected against silting and draining freely, the stress is taken as 0.4 kg per sq cm.

3. Ice saturation is determined from the formula I = W/Wp, where W is the weight of ice (water) in the ground, and Wp is the water-holding capacity of the thawed ground.

4. In checking the tensile strength of the foundations in accordance with the adfreezing stresses given in the table, the ultimate tensile strength may be used, without any safety factor.

permafrost is absent (most advantageous location). A road structure should not be constructed over imbedded ice, since this would result in melting of the ice and deformation of the structure. River crossings should be located at points where there is only one deep channel running between high banks and containing no island or former beds. It is inadvisable to locate crossings in the vicinity of sandbanks because river icings tend to form in such areas, which consititutes a hazard to the bridge and its approaches.

Long diversion ditches, which accommodate the water from several channels and direct it towards a bridge opening or culvert, are inadvisable in permafrost regions. Such ditches are permissible only in solid rock and when the flow is fairly small. Long ditches are inadvisable for several reasons. Firstly, every ditch in the permafrost disturbs the existing permafrost regime. Secondly, it is difficult to maintain the drainage ditches, even if they have shallow grades. Finally, proper stability of the ditches cannot be achieved under permafrost conditions. It is essential to avoid deep drainage ditches in order to lessen the negative effect of the water on the stability of the permafrost regime. For the same reason, it is inadvisable to deepen the channel of a stream near a small bridge if the bridge is to be designed in accordance with the principle of permafrost conservation at the foundation base.

The openings of large and medium bridges or culverts are determined from results of ordinary hydrometric investigations. The openings of small bridges may be determined in accordance with local engineering conditions, with due consideration of the climatic and meteorological conditions of the given region and the factors influencing the rate of flow. Since accumulations of large masses of water greatly affect the permafrost regime, the opening beneath a bridge should be of sufficient size so as to prevent upstream accumulation of large quantities of water for long periods of time.

It is important to note that designing the size of an opening beneath a small bridge under permafrost conditions is extremely complicated because the ground (except for mari), which does not thaw during a definite part of the year and is supersaturated during the remaining part, does not readily absorb water, and because it is difficult to estimate the various

rates of flow, since these rates depend on the surface cover which varies greatly from one year to the next. It is further necessary to realize that no data are available regarding the regime of rivers in the permafrost region, even in the case of large rivers. It is obvious, therefore, that no information is available regarding the hydrometric aspect of small waterways and streams. Therefore, although the discharge is an essential factor in the design of a bridge, the only available data would be those obtained from observations carried out during the two or three years from the initial studies to the beginning of construction. Of course, these data are inadequate for good design. Accordingly, it is perhaps advisable to construct temporary wooden bridges in places for which definite data are not available and where local conditions make it impossible to obtain adequate data for use in analysis using standard formulas. Exceptions are allowed in cases where temporary wooden structures are not feasible because of special considerations. While these bridges are in use, the passage of flood waters beneath these bridges can be observed over a period of several years, making accurate data available for the design of the bridge opening and for selection of the proper type of permanent structure.

The size of the opening beneath a small bridge and the sizes of individual spans of medium and large bridges are additionally influenced by the fact that rivers and streams in the permafrost region, which pass through forested areas, carry a large number of uprooted trees during high stage. These trees may block and clog small spans. On the other hand, experience has demonstrated that piers of small bridges with maximum spans of 20 to 25 m are deformed more frequently than piers of larger spans. Moreover, the smaller the span the more extensive is the deformation. Accordingly, it is advisable to avoid small spans when this is feasible with respect to engineering and economic consideration. The minimum span of a multispan bridge should range from 21 to 23 m in the case of steel structures and from 12 to 14 m in the case of reinforced concrete structures. These magnitudes are determined in accordance with existing typical multispan bridges and from considerations of the weight of the piers and the loads on them. The difference in minimum span length of steel and reinforced concrete structures is based on the fact that weight of and the load on an individual pier should be approximately

the same in both cases, per unit bearing area of the piers.

It is advisable to avoid multispan bridges with small spans. Single span bridges on abutments are preferable. Of course, this should be done only when a single span bridge is not excessively costly or is not feasible because of complications in constructing the approaches. Under the complicated conditions of permafrost, however, it is perhaps judicious to erect a relatively more costly structure, but such that would not deform. Experience in construction of bridges on permafrost has not presented a single case of deformation of such abutments. With regard to bridges with large spans, it has been noted that such bridges are not subject to deformation; their piers do not heave, settle, or tilt.

Selection of the proper type of road structure depends on local conditions, economic considerations, and the regime of the frozen ground. These structures comprise the following; filter dams, culverts, and bridges erected on piles, cribs, or solid piers.

Filter dams may be used in the case of dry beds if the dams are protected against silting and clogging with debris. In this case they may replace culverts or small bridges if the expected flow is 5 to 10 cu m. These dams are recommended not only on account of the usual engineering and economic considerations, which are influential factors in each instance, but also because use of such dams on a certain major road has been quite satisfactory. However, the data regarding their effectiveness are not definitive because the dams were constructed not long ago. Therefore, such dams may be used in place of bridges or culverts only if they satisfy the series of requirements listed later. These requirements may be modified in the future when additional and more definite information about the behavior of filter dams is available.

Culverts are not suitable in areas where icings may form. The culvert diameter should be not less than 2 m in the case of wet beds or streams in which flow occurs also in the alluvium of the bed. It should be noted that culverts deform as readily as bridges, but their deformation does not effect the roadbed and interfere with its use to the same extent as the deformation of bridges.

If it is impossible to determine accurately the bridge opening because of inadequate information regarding the region, the watershed, and the flow regime involved, it is advisable to build wooden bridges in accordance with the principles discussed previously. Wooden bridges on piles may be erected on permafrost or in areas where permafrost is absent. However, the essential condition is protection of the piles against heaving. Cushioned abutments are recommended in the case of reinforced concrete or steel bridges erected on ground that is subject to considerable heaving. Masonry bridges are suitable only if they can be based on rock.

Bridges on solid piers are suitable in places where the bedrock occurs at shallow depth; however, wooden bridges on crib piers should be used if it is not feasible to calculate the required opening. Wooden bridges on ordinary timber grillages are not suitable in permafrost regions because of the extensive and inevitable deformation involved. Exceedingly complex and expensive measures are required to prevent deformation of such bridges. Where the permafrost is merging or layered, wooden bridges may be erected on piles driven by means of steam points or on crib piers. If accurate computation of the openings is feasible, bridges on solid foundations may be used under these conditions.

2. Design of Filter Dams

Filter dams are made of stone. Their dimensions and shape are determined by hydraulic analysis. Figure 104 is a general view of such a dam designed for a recently built road. This shape is preferable to that shown in Fig. 105 because the latter tends to deform during spring thawing of the frozen slopes. The deformation is due to the weight of the stones on the underlying layers. The stone used in a filter dam should be frost-proof. This property should be established by means of suitable tests using repeated freezing and conducted in accordance with standard methods. The size of the stones should be relatively uniform. It is recommended to use stones measuring not less than 30 cm in diameter and weighing 40 to 80 kg. The size of the stones should be related to the hydraulic analysis. A stepped foundation should be used in the case of a filter dam erected on slopes. The ledges should be filled with dry masonry over moss.

The following is a rational procedure for construction of a filter dam. Stones of relatively uniform size are laid in transverse rows. The gap between adjacent rows is 5 to 10 cm and is bridged with an upper course of stones. The stones are laid flat and are well fitted, each stone being placed in the most stable position possible, so that the stonework resembles a honeycomb. The dam should be raised to the height of the expected water level, but should contain not less than four courses. A 50-cm layer of small stones, sufficiently large not to sift into the spaces between the underlying large stones, is placed on top of the upper layer of stonework. This layer is covered with 50 to 60 cm of fine pebbles or gravel. This layer of gravel is overlain by a layer of moss or peat 25 to 30 cm thick. The uppermost layer, extending to design elevation, consists of regular material used in a road. The dam sides along the road are also covered with 25 to 30 cm of moss or peat.

The stream bed near the filter dam should be paved in accordance with the design velocity of flow and local conditions. A fence made of stones, brushwood, or fascines is used to prevent silting of the filtering portion of the dam. Filter dams are not suitable in areas where formation of icings is possible.

3. Design of Culverts

Culverts may be built of reinforced concrete, ordinary concrete, or masonry. The choice of material depends upon local availability, consideration being given to normal engineering requirements and special permafrost conditions. It is recommended to use culverts with tapered collars and straight wingwalls. Reinforced concrete culverts should be used if the ground is subject to swelling, that is, when it does not consist of sand or gravel.

The foundations should have a rectangular horizontal section. In the case of an embankment less than 5 m high, the culvert foundation should widen toward the base so that the sides would slope at an angle not greater than 80° (Fig. 106). These sides should be smoothly finished and coated with crude oil, pitch, tar, or a similar substance. If the embankment is over 5 m high, such foundations are required only under the collars and the

extreme members of the culvert, while the remaining portion of the foundation may be of standard design. If the embankment is less than 3 m high and is erected on permafrost, the foundation of the entire culvert should rest on a grillage consisting of two crossed layers of wooden beams measuring 16 by 16 cm. The grillage should be placed on the thinnest possible layer of clean sand used merely to level the bottom of the excavation. If the embankment is more than 3 m high, grillage may be used only under the extreme members and the collars of the culvert.

If the ground is excessively wet and subject to swelling, it is advisable to remove the local ground around the foundations beneath the collars and the extreme members and to fill the space with coarse gravel (5 cm in diameter) soaked in fuel oil. The width of the backfill should be not less than 1 m. It is recommended to protect the backfill against sifting sand. If the embankment is low, the gravel backfill around the foundations should extend the entire length of the culvert.

If permafrost is absent or occurs at great depth, the depth of the culvert foundations is determined in the regular manner. When permafrost occurs directly beneath the active layer, the foundation depth under the collars and extreme members should be equal to the thickness of the active layer (determined on an area from which the surface cover has been removed) plus a 1.0 to 1.5 m extension into the permafrost. This depth includes the height of the grillage and the underlying sand. In the case of concrete or rubble foundations, the foundations beneath the center members of the culvert may be extended into the permafrost only to a depth of 0.5 m. If the bottom of the culvert consists of a reinforced slab about 0.5 m thick, the foundation beneath the center members may be shallower. In this case, however, the grillage should extend beneath the entire culvert, regardless of the height of the embankment.

The function of the timber grillage beneath culvert foundations is twofold: (1) it is not heat conductive, so that it prevents thawing of the ground beneath the foundation during construction, and (2) it facilitates uniform distribution of the load on the base, thus improving the capacity of the structure to resist deformation when the ground thaws. The grillage constitutes a heat insulator to a certain extent.

Backfilling culvert foundations with gravel is recommended as a supplementary measure of protection against heaving of the foundation. Despite the fact that construction engineers recently began to regard such backfilling as an ineffective measure against heaving, nevertheless it is valuable. Of course, if such backfilling is difficult or costly, sand or slag may be used, or it may be entirely omitted. It is recommended not to remove the foundation forms, but to leave them in the ground.

4. Design of Timber Trestles and Bridges

The choice of design for a timber trestle or bridge depends on local conditions. The following criteria should be applied. Girder and truss systems of wooden bridges may be used if the permafrost occurs at great depth or is entirely absent in the given area and if the ground contains little moisture and does not swell, as in the case of gravel or coarse sand. Truss systems and particularly multiple truss systems cannot be used if the active layer is excessively wet, even though it consists of sand, regardless whether permafrost is present or absent. Only girder bridges supported on piles are permissible if the active layer is wet and consists of clay loam, sandy loam, loamy sand, or fine sand.

The limitation imposed upon the use of truss bridges is due to the fact that relatively small heaving or settling of the support of a truss bridge inevitably causes deformation of the structure and excessive stresses in individual members. The resulting deformation interferes with traffic across the bridge and sometimes involves complete destruction of the bridge.

If the permafrost occurs at shallow depth immediately below the active layer, it is permissible to use girder bridges resting on piles driven into the ground by means of steam points, as well as girder bridges on low cribs. The proper procedures to decrease the strength of adfreezing between the active layer and the pile are presented later.

Wooden bridges are not recommended if bedrock occurs at shallow depth. If they must be used, they should be supported on cribs or stone piers. Flat bases of the usual type should not be used for trestles and

bridges in the permafrost region if the ground is wet, because of the excessive deformation involved. They should not be used also if bedrock occurs at shallow depth. Although piles cannot be used if the bedrock is near the ground surface, nevertheless use of flat bases under such conditions is difficult because it is necessary to employ special devices to anchor the footings to the rock, which is complicated, costly, and unreliable.

Wooden bridges, trestles, and other structures erected on piles in the permafrost region do not differ from similar structures erected under normal conditions, except that the pile foundations are arranged in accordance with several particular factors. It is necessary to consider the following two possibilities: (1) the piles are driven into thawed ground, that is, permafrost is absent, and (2) the piles are sunk into the permafrost layer. In both cases the piles may be subjected to heaving due to swelling of the active layer. The possibility of heaving is greater when the ground consists of fine-textured material, and is somewhat smaller if the ground consists of well-draining material. In general, however, the determing factor is the moisture content of the ground. Fully saturated or supersaturated ground is the most hazardous in this respect.

Where permafrost is absent or occurs at great depth, the piles should be driven to at least twice the depth h of the active layer plus 1.0 to 1.5 m (Fig. 107) and should be tested for resistance to heaving on the basis of the fact that the heaving force N must not exceed the resisting force S. The latter force is determined by means of standard computation of the bearing capacity of piles 15, 20 in various types of ground. This computation is based on the fact that a pile is retained in the ground by friction along the depth

$$h_1 = h + 1.0 \text{ to } 1.5 \text{ m}$$

In other words, it is assumed that the pile is held by the ground layer h_1 below the active layer. In computing the forces involved, it is recommended to take into consideration the fact that a certain fraction of the heaving

force N is counteracted by the weight of the pile and the constant load the pile, which constitute the magnitude Q. Accordingly, the condition stability of the pile is

$$N < S_1 + Q$$

in which S_1 is the force depending on the firmness with which the pile held in the layer of height h_1.

The allowable total vertical load is established by means of driving tests and by corresponding standard computations. This load sh not exceed the magnitude established by specifications.

If the pile is extended into the permafrost, as shown in Fig. heaving of the pile is counteracted by the adfreezing between the pile the frozen ground of depth h_1. Therefore, the depth h_1 necessitates com putation, but should exceed the thickness of the active layer by 1.0 to 1.5 m. It is essential that the heaving force on the pile should not ex ceed the adfreezing strength between the pile and the permafrost layer depth h_1. A constant load on the pile may be assumed. The condition fo stability of the pile is

$$N < h_1 \, p \, t + Q$$

in which p is the perimeter of the pile, t is the magnitude of the stres determined in accordance with OST Tables III and IV, and Q is the consta load on the pile, including the weight of the pile. In those computatio the thickness of the active layer should be determined at the constructi site under conditions of bare surface. It is advantageous to drive the pile with the butt end downward, since this increases their resistance t heaving to some extent. It is inadvisable to splice piles within the ac layer because heaving may sever them (Fig. 32).

In order to reduce the adfreezing strength between the active layer and the pile, the following measures are recommended for use in suitable cases. The section of the pile within the active layer should

smoothly planed and coated with tar, pitch, fuel oil, or naphtha, after all the cracks have been sealed. It is advisable to use a peat cushion 30 cm thick and 2 m in diameter around the pile at the top, and a covering layer of local ground. These measures may be omitted in the case of piles set within the cone of the fill and protected by a ground layer 2 m thick as well as in the case of piles located either in a nonfreezing stream bed or in a freezing stream the bed of which consists of well-draining material (gravel, and clean, coarse or medium sand).

Crib piers may be used to support the bridge proper or as bases for the support framework. The cribs should be built as follows. The outside corners of the underground portion of the crib should dovetail and the corresponding lower courses of timbers should be trimmed to form a single edge which is faced with planks. This arrangement provides a relatively high smooth surface. Longitudinal and transverse timber braces should be installed inside the crib and fastened to the corresponding sides. The rows of timbers located in the ground should be fastened together with vertical bolts 20 mm in diameter and placed at an interval of 0.75 to 1.0 m. The vertical clamps inside the crib should extend from top to bottom. Exterior clamps are necessary only for the part of the crib above the ground.

When cribs are built, the entire active layer down to the underlying permafrost, rock, or thawed layer, should be excavated and backfilled with rubble or gravel averaging 5 to 10 cm in diameter. The pit should be sufficiently larger than the crib, so that the backfill would have a slope not greater than 45° (Fig. 109). The slopes of the gravel fill should be covered with a 30-cm layer of moss or peat and the remainder of the pit should be backfilled with coarse gravel mixed with peat or moss. The latter backfill may consist of slag or finer gravel. The bottom of the crib should be located a maximum of 70 cm below the ground surface. The surface of the gravel fill beneath the crib should be leveled with small stones covered with a layer of moss or peat and a layer of fine gravel or coarse sand. The bottom timbers of the crib rest upon the latter layer.

The crib should be filled with stones or rubble of assorted sizes. Coarse or fine gravel and even sand may be used in order to increase the

weight of the crib. If the crib rests on permafrost, it is necessary to place within the crib layers of moss, peat, straw, or some other insulating material, each 20 to 30 cm thick, at vertical intervals of 0.50 to 0.75 m. It is essential to take measures to reduce the strength of adfreezing between the crib and the ground and to make provisions for regulating the elevation at which the structure would rest on the crib.

5. Design of Solid Supports

Deformation of solid supports is due primarily to thawing of the permafrost layer at the base of the supports and to swelling of the active layer. Therefore, the design of such supports depends upon the permafrost conditions at the construction site as well as the texture and moisture content of the ground. To facilitate the analysis of these aspects, and in order to coordinate properly the various designs and layouts with the feasible construction conditions, the various types of design are classified into several categories and the most suitable type of support is indicated in each case.

a. Solid Bridge Supports Where Permafrost Is Absent or Occurs at Great Depth

Solid supports of any type and spans of any length, except wall piers which deform readily and extensively, are permissible when permafrost is absent or occurs at great depth, when the active layer consists of well-draining material such as rubble, gravel, coarse or medium sand, and when the structure can be based on rock. Small bridges (with a span of 21 to 23 m, or with a span of 12 to 14 m in the case of a reinforced concrete structure) without intermediate piers are preferable when the ground consists of fine-textured material with high moisture content. Under these conditions, use is recommended of single-span bridges on abutments, preferably cushioned abutments (Fig. 110).

Multispan bridges with small spans (less than 23 m, or less than 12 m in the case of reinforced concrete structures) may be used when the piers are located in a stream where the minimum depth of flow is 2 m, even if it freezes in the winter, as well as when the ground consists of well-drained material. In the case of ground consisting of loam and fine sand,

particularly if it is extremely wet and silty and tends to turn into slud, center piers of small bridges (with spans less than 23 m and 12 m, respectively) are permissible only when it is definitely impossible to use a single span bridge. The piers should be designed to withstand the heaving stresses (in accordance with the instructions presented in Chapter II, B-1). Similar piers for such bridges may be located on freezing river beds consisting of fine-textured material, as well as on floodplains. Piers located in a nonfreezing stream may be of arbitrary design, but preferably without an ice deflector or with an ice deflector of minimal length. Bridges with spans exceeding 55 m, erected on any type of ground, may utilize supports of arbitrary design, except wall piers.

Cushioned abutments, previously recommended for small bridges are of usual design. However, the part of the abutment within the active layer should be made of concrete or rubble concrete. The front and side faces of the abutment foundation should have a maximum slope of 80°. The rear face may be vertical (Fig. 110). In order to reduce the strength of adfreezing between the active layer and the abutment, the foundation surface within the active layer should be smoothly finished with cement mortar. The abutment above the foundation level may be built of rubble, concrete or rubble concrete. The foundation depth is determined in accordance with the usual considerations. The abutments for bridges with spans exceeding 23 m, or exceeding 12 m in the case of reinforced concrete structures, do not have to be cushioned if the ground is not subject to swelling. The horizontal section of piers for such bridges, located on a floodplain or in a shallow stream which freezes to the bottom during the winter, as well as on swelling ground, may be rectangular with corners rounded to a radius of 30 cm or may have triangular prow and stern rounded in a similar manner (Fig. 111). To protect the piers from the effect of swelling of the active layer, it is advisable to taper the side faces of the pier within the active layer at an angle of 80°, to make them smooth, and to coat them with cement mortar. It is advantageous to use a gravel backfill around the pier.

The pier, exclusive of its foundation, may be built of any of the materials normally used for piers. If considerable heaving force is expected,

the part of the pier located in the ground should be made of concrete or rubble concrete with vertical reinforcement to protect the masonry against rupture due to swelling of the ground. The reinforcement is arranged along the external faces of the foundation and is connected with stirrups. The reinforcement should be calculated in accordance with Chapter II, B-1.

To avoid heaving, piers for bridges with small spans (less than 23 m, or less than 12 m in the case of reinforced concrete structures) should be erected on piles or sunk wells if the piers are located on a flood plain, if the active layer is more than 2 m thick, if the active layer consists of silty or slud-like material which tends to swell extensively, and particularly in the case of low embankments that are less than 5 m high. The pile system should be arranged in such a way that swelling of the active layer would not force the piles out of the ground and that the cohesion between the piles and the concrete foundation would not be disturbed. This should be verified analytically. In addition to driven piles of timber or reinforced concrete, use may be made of concrete piles poured in place (Strauss piles, for example) and reinforced with longitudinal bars which take up the tension occurring during heaving. However, such piles may be used only when the ground temperature at the base of the pile is not lower than + 5° C; when the temperature is less than 5° C, but not lower than 0.5° C, these piles may be used if proper measures are taken to assure adequate hardening and setting of the concrete.

The piers of bridges with spans exceeding 55 m may be of arbitrary design.

The following specifications are recommended for pile foundations and sunk wells. The longitudinal reinforcement of driven piles of reinforced concrete should extend into the pier foundation to a depth of 30 d, where d is the diameter of the reinforcing bar. After the pile has been driven, the pile head should be sheared off so as to expose the reinforcement. The head of a timber pile should penetrate at least 1 m into the concrete footing, and this section of the pile should be notched to a depth of 2.5 cm at intervals of 20 cm. If the piles are connected

with tie beams bolted into notches, the required depth of penetration is only 60 cm. The allowable bond between concrete and wood may be taken as 5 kg per sq cm.

When reinforced concrete and timber piles are driven by means of steam points and the pile foundation is designed to resist heaving of the piers, after the thawed ground has refrozen, the resistance of these piles to heaving and their bond with the foundation should be computed analytically. This computation should be made in accordance with the specifications presented in Chapter II, B-1.

Masonry sunk wells are not suitable. Wooden or reinforced concrete sunk wells are preferable. Such wells are designed in the usual manner. Supplementary calculations are required only to verify the wall stresses occurring due to heaving of the supports in the case of a thick active layer. The tensile stress in the walls of the well is determined in accordance with the considerations previously presented in this chapter. The following recommendations are essential for the design of wooden sunk wells. The well should be faced with vertical planks both inside and outside. Steel tie rods should extend from the bottom to the top of the well. These rods should be able to resist the heaving force which has not been counteracted by the weight of the support and the constant bridge load.

b. Solid Supports for Bridges on Permafrost

When it is necessary to construct solid bridge supports on permafrost, it is recommended to use single span bridges with cushioned abutments, as in the preceding case. If the bridge opening does not exceed 23 m, or 12 m in the case of reinforced concrete structures, center piers are permissible only by way of exception. When it is necessary to construct such piers on frozen ground in floodplains, the piers should be designed in accordance with the specifications presented in the following under (b). The pier design shown in Fig. 111 is suitable for piers located in a stream in which the water is not less than 2 m deep, regardless of whether or not it freezes in the winter.

It is recommended to support the center piers of bridges with small spans (up to 23 m, or up to 12 m in the case of reinforced concrete

structures) on piles or on sunk wells, either wooden or of reinforced concrete, if it is assumed that the supports are likely to heave due to the presence of a relatively thick active layer consisting of silty material. Presence or absence of permafrost is immaterial in this case. Pile foundations and sunk wells may be used as supports for bridges with small and medium spans even if the supports rest on a permafrost layer occurring at shallow depth, or when intensive degradation of the permafrost is possible. Sunk wells for such bridges are advantageous even when rock occurs at shallow depth beneath the ground surface, regardless of the presence or absence of permafrost in the given area.

Supports for bridges with spans exceeding 55 m may be of arbitrary design if they are sunk at least 6 m below the ground surface. Cushioned abutments are advisable if the depth of the supports is smaller. The design of abutments and piers will be examined here separately because each of these types of supports has certain special characteristics which should be taken into consideration in order to achieve the best design with respect to deformation.

1.. Design of Abutments

As stated previously on several occasions, the cushioned abutment is the most suitable type of abutment under permafrost conditions. This abutment is completely covered with fill material, so that the probability of thawing of the permafrost at its base is negligible. In the case of a thick permafrost layer which is conserved at the base of the supports, cushioned abutments for small bridges (with spans of about 21 to 23 m or less in the case of steel bridges, or up to 12 m in the case of reinforced concrete) should be constructed as follows. Erection of foundations on the assumption of permafrost conservation at the base of the structure is permissible only when the permafrost temperature at the base of the foundation is not higher than -0.5° C during maximum summer thawing of the ground. The abutment should extend into the permafrost to a minimum depth of 1.5 m and should ~~rest on a~~ grillage consisting of two crossed layers of green timbers measuring about 16 by 16 cm. The grillage should be overlain by a concrete or rubble concrete slab of rectangular horizontal section and

vertical edges. The thickness of this slab should equal the depth to which the foundation penetrates into the permafrost (Fig. 110). The horizontal dimensions of the foundation are determined in the usual manner in accordance with the allowable stresses presented in Chapter IV, B-2d.

If the permafrost is undergoing degradation and cannot be conserved at the foundation base, the cushioned abutments may be designed as above but the foundation should extend into the permafrost to a depth not exceeding 1 m, provided the permafrost does not consist of silt, loam, or slud-like material, and provided it contains no more than 30 per cent moisture and no ice wedges. In such a case, the horizontal dimensions of the foundation are determined in accordance with allowable stresses for thawed ground, without taking the permafrost into consideration.

In addition, the abutments of double track bridges erected under these conditions should be analyzed for transverse bending. Accordingly, it is necessary to reinforce either the top or the bottom of the abutment in order to avoid failure of the masonry due to nonuniform thawing of the permafrost. The analysis is carried out as presented later. Such abutments can be built without reinforcement if each abutment is built of two individual parts separated by a vertical joint as shown in Fig. 112.

In the case of weak permafrost consisting of silty or slud-like material with high moisture content and ice wedges, the abutment should rest on a pile foundation or on sunk wells lowered into the permafrost by means of steam points. The abutment should be exceptionally firmly attached to the piles if the active layer in this case is subject to swelling, consists of silty or slud-like material, is excessively wet, and is more than 1.5 m thick.

In the case of layered permafrost, it is recommended to penetrate through the upper layer or layers of the permafrost if they are relatively thin and to support the abutment on the underlying layer of thawed ground or on one of the lower, sufficiently thick permafrost layers. Construction of the support foundation is carried out as in the previous case.

In order to conserve the permafrost where its regime is stable, it

is recommended to carry out construction operations during the winter. For the same reason, it is essential to cause minimum disturbance of the natural regime of the area near the abutment, conserving the surface and vegetation covers.

Where bridge spans exceed 23 m, or 12 m in the case of reinforced concrete structures, the cushioned abutments may be of usual design if they extend more than 6 m below the ground surface, and no special measures are required except a rock fill around the foundation within the active layer and an insulating layer of peat or moss placed on the cone of the fill. At lesser foundation depths, it is necessary to take measures to protect the abutment against heaving.

2. Design of Piers

Center piers of bridges with spans less than 23 m, or less than 12 m in the case of reinforced concrete structures, erected on a thick layer of stable permafrost may be designed in accordance with the following specifications. The pier foundation should extend at least 2 m into the permafrost and should rest on a timber grillage, as in the case of the abutment. A cushion of rubble with layers of moss or peat should be placed on the ground surface around the pier. The horizontal dimensions of the foundation are determined by means of usual calculations in accordance with the allowable stresses presented in Chapter IV, B-2d.

In the case of degradation of the permafrost, if the permafrost beneath the base of the support is neither silty nor slud-like and contains neither ice wedges nor more than 30 per cent water, the pier need be extended into the permafrost to a depth of only 1 m. If the ground at a depth of 1 m does not satisfy these requirements, but satisfactory ground occurs somewhat lower, the foundation depth should be changed accordingly. The horizontal dimensions of the foundation should be determined in accordance with the allowable stresses for ordinary thawed ground, without taking the permafrost into consideration. Piers for double track bridges under similar conditions should be designed to resist transverse flexure or should be made of two separate parts (Fig. 112). If the permafrost is in the process of degradation and consists of unstable, silty or

slud-like material, and contains much water as well as ice wedges, it is advisable to erect the piers on pile foundations or sunk wells, in accordance with the corresponding specifications presented previously.

If the permafrost is layered or is in the process of degradation, it may be advisable to construct the piers on thawed ground after the permafrost has been eliminated by means of steam points in accordance with the considerations presented in Chapter IV, B-4. This procedure is judicious only in suitable cases and when it is relatively economical. The foundation dimensions are determined in this case in accordance with the allowable stresses on thawed ground. If the active layer is thick and the ground is fairly wet, the pier design should be checked with respect to resistance to heaving.

Piers of bridges with spans exceeding 23 m (or 12 m in the case of reinforced concrete structures), sunk more than 6 m into the ground, require no special design features if the foundation extends at least 2 m into the permafrost layer.

3. Remarks Regarding the Design of Abutments and Piers

Support foundations resting on permafrost having a temperature near zero and not lower than -0.5° C should be designed on the assumption that the permafrost may thaw. Accordingly, the area of the base if determined in accordance with the allowable stresses for the thawed ground at the given site beneath the given structure. This determination is experimental. However, preliminary calculations may be made on the basis of the allowable loads presented in Chapter IV, B-3. Since it is impossible theoretically to determine the allowable load on loam or loamy material in the case of corresponding thawed permafrost, it is recommended to use piles or sunk wells because such thawed ground settles extensively and has negligible bearing capacity.

In view of the fact that the permafrost may thaw nonuniformly, abutments and piers for double track bridges should be designed to withstand flexure on the assumption that (a) the support is a simple beam, as shown in Fig. 113, and (b) the support is a double cantilever beam, as shown in

Fig. 114. If either calculation shows that the tensile stress in the masonry exceeds the allowable stress, it is necessary to reinforce the top or bottom of the support. The reinforcement is determined as in the case of ordinary reinforced concrete or reinforced masonry construction. Reinforced supports of concrete or rubble concrete are preferable.

In addition to the aspect of flexure, the support should be designed to withstand heaving and tension in accordance with Chapter II, B-1, and Chapter IV, C-4. Solid masonry supports, both abutments and piers, erected on permafrost in accordance with the principle of permafrost conservation, may be calculated as in the case of nonpermafrost conditions, provided the permafrost temperature at the foundation base is not above -0.5° C. The allowable load is determined from field tests of the strength of the frozen ground. Preliminary estimates may be made in accordance with OST Table II.

c. Tunnels

Tunnels are designed and calculated as under ordinary conditions, without taking the permafrost strength into consideration because construction of the tunnel causes destruction of the permafrost in the immediate vicinity of the tunnel. Proper waterproofing is highly essential in the severe climate of the permafrost regions because the water which seeps through the facing into the tunnel causes major complications both during construction and during subsequent operation of the tunnel. The importance of adequate waterproofing is also due to the fact that ground water in permafrost regions often is highly aggressive. Accordingly, tunnel facings should be protected with waterproofing installed outside the arch. If this is not feasible, the waterproofing may be installed inside the tunnel if underlain by a reinforced concrete lining.

In order to remove the water reaching the tunnel facing, it is advantageous to install longitudinal drainage galleries along both sides of the tunnel. If they connect to the facing, they should be accessible from within the tunnel for inspection, cleaning, and repair. However, if they are installed independently and are connected with the tunnel, they should be of adequate dimensions to permit passage of a workman. These

galleries should have suitable facing.

During facing of tunnels with masonry or concrete, the tunnel section to which the facing is applied should be maintained at a temperature adequate for proper hardening and setting of the concrete (not lower than 5° C); this temperature should be maintained until the mortar or concrete attains its design strength. Use of high-grade cement and a rapid rate of work would shorten this time interval.

D. Specifications and Factors Pertaining to Stable Earth Structures

1. Fills

The stability of a fill depends primarily on the material involved as well as the nature and state of the base, that is, the ground beneath the fill. Therefore, particular attention is given here to these two aspects, and the following is specified: the best materials for a fill of any height, the undesirable materials which may be used only under definite limited conditions and ~~subject to~~ special measures, and the materials that are definitely unallowable.

A fill of well-draining material, such as coarse or medium sand, gravel, or rubble, may be of any height desired, provided the base is sufficiently strong and the steepness of the slopes is in accordance with the standard for the material involved. Therefore, the following fill materials are recommended: coarse or medium sand, gravel, pebbles, rubble, and rock removed during blasting of cuts. Fills of such materials usually do not deform.

Fine sand, loam, and loamy ground are less suitable for fills, but may be used if a special base of well-draining material is provided. This base prevents penetration of water (the most detrimental factor) into the core of the fill. Silty material, sandy loam, and loamy sand containing 30 to 35 per cent silt are not recommended for use in fills. They may be used in exceptional cases when it is impossible to furnish more suitable materials and under condition that the silt fraction does not exceed 50 per cent. Loamy sand and sandy loam containing more than 50 per cent silt as well as extremely fine textured or sludge-like material are not suitable

for fills. When such material is used because there is no alternative, the fill should be of special design.

Fills of fine sand, sandy loam, and clay loam may be constructed in the following cases and under the following conditions: (a) when the fill is up to 1 m high and is located on dry ground; (b) when the fill is 2 to 5 m high it may be located on either dry or wet ground, except mari or other swamps, but the slopes should be not steeper than 1:1.5 and the fill should rest on a specially prepared base of well-draining material. The design of such a base is discussed later because this type of base is applicable in numerous other cases.

It is evident from the foregoing that transition fills (where the roadbed proceeds from a cut to a fill), as well as all fills which are up to 1 m high and consist of fine sand, clay, and loam, may be built only in dry areas. This specification arises from the fact that water from a wet base would tend to rise high into such a fill because of the high capillarity of clay loam. This may cause swelling due to deep freezing of the fills in the permafrost region, resulting from exceedingly low winter temperature and aggravated in some cases by inadequate snow cover.

Those considerations evolve the need to protect the lower part of a high earth fill (2 to 5 m) against saturation with water. Accordingly, it is recommended to construct a special base of well-draining material. Fills between 2 and 5 m high are not thick enough to maintain their bases in a permafrozen state. As a result, the lower portion of such a fill would ultimately freeze and thaw, and when thawed and saturated it would tend to be squeezed out from beneath the fill. Fills over 5 m in height are not as greatly affected by alternating freezing and thawing of their lower portions because this phenomenon would occur only near the slopes which can be prevented from sliding by means of surface reinforcement. The center part of the lower portion of the fill would probably remain frozen because the high fill prevents heat transfer to the ground and thus raises the underlying upper permafrost limit. When the fill is sufficiently high, this limit may rise into the fill itself in the form of a longitudinal hump. Nevertheless, it is advisable to provide a special base of well-

draining material even in the case of high fills if they consist of clay loam. This recommendation is based on the following considerations. Firstly, when a fill is erected in a wet area, the lower portions of it slopes, corresponding to the depth of summer thawing, are subject to conditions analogous to those of low fills and, consequently, may tend to slide. Secondly, the lower portion of the fill core may be subject to the same conditions during several years until the upper permafrost limit rises into the fill proper. The opposite occurs during construction and several years afterwards—the upper permafrost limit beneath the fill recedes to some extent and a trough of weak ground saturated with water is formed beneath the fill. Such cases have actually occurred; they were caused by the extensive heat stored in the fill material during summer construction when the sun heated the surface of the fill material to a considerable extent (Chapter II, D-2).

In the case of fills over 5 m in height and consisting of earth material, use of a special base of well-draining material is insufficient, and it is necessary to construct berms on both sides of the fill. The need for berms is explained as follows. As stated previously, the permafrost beneath high fills tends to rise into the fill and form a hump. This phenomenon is readily explained theoretically and was substantiated by observations. When the ground thaws, wet and steep sliding surfaces are formed at the contact area between the thawed ground and the permafrost hump. These surfaces may cause deformation of the fill. To avoid the occurrence of the steep frozen hump in the fill, it is necessary to cause the upper permafrost limit to rise along the sides of the fill as well. This is achieved by means of berms constructed along both sides of the fill. Of course, the width and height of these berms depend on the height of the fill. The height of the berms may be reduced if they are made of moss or peat. However, the use of earth material is advantageous because it has insulating properties and its weight increases the stability of the fill slopes.

The recommendations with regard to loamy ground pertain to silty ground as well, except that the latter ground is even more hazardous when saturated and fills comprising silty material deform more readily and

extensively. Therefore, fills consisting of ground containing 30 to 50 per cent silt are permissible only in exceptional cases under condition that the fill is protected against saturation with water. The slopes of low fills (up to 2 m) of silty material should be 1:1.5. In the case of higher fills, the upper 2 m may have a slope of 1:1.5, while the lower portions should be increasingly flatter, at a rate of one-fourth for every 3 m. The stability of such a fill should be verified analytically.

Occasionally it is necessary to construct fills in the permafrost region on mari. This often results in deformation of the fill because the underlying ground is exceedingly wet and the permafrost occurs at shallow depth. Stability of such fills necessitates the following special measures. The relationship between the height of a fill on mari and the type of material used is as follows: fills of stone, rubble, or coarse gravel should be not lower than 1 m; fills of fine gravel and coarse or medium sand should be not lower than 1.5 m; fills of other suitable material should be not lower than 2.0 m and should have a special base of well-draining material. The thickness of this base layer should be determined in accordance with the probable settling of the underlying peat due to compression, that is, the draining base should rise at least 1 m above the surface of the mar. A lesser thickness is inadvisable because the base may readily and rapidly become silted due to sifting of fine fill material from above and rise of water from below. The height of a fill on the mari should never reach zero as may be the case along a dry stretch, not should it ever be lower than specified previously. This condition can be readily satisfied if the routes over mari are properly planned. The transition from fill to cut should occur at the edge of the mar and at a suitable distance from it, so that the transition fill within the mar would attain the required height. A height of 1 m should be regarded as the minimum height which would prevent rapid thawing of the permafrost beneath the fill, since the permafrost in this case occurs at shallow depth.

Observations demonstrated that fills about 1 m high prevent any rise of the permafrost level beneath the fill. Moreover, the permafrost usually thaws to some extent, which results in settling of the fill even

in cases when the moss and peat covers of the mar beneath the fill have been conserved. Since the peat beneath the fill on a mar (it is perhaps advisable not to remove this peat) gradually compresses under the weight of the fill and the trains, the lower portion of the fill tends to sink into the mar. Such settling may attain 2 m. Since a peat layer saturated with water may compress to one-quarter of its original thickness, the entire low fill over a mar should consist of well-draining material, although only a base of such material is required in the case of fills constructed on other types of ground. In the case of a fill exceeding 3 m in height, the thickness of the special base consisting of well-draining material should be determined in accordance with the probable settling, so that the layer of well-draining material would extend at least 1 m above the surface of the mar.

Experience with railroad fills built on mari proves that even minor drainage installations may change the swampy aspect of the mari. Melioration of mari occurs as follows. As the ground-water level is lowered, the uppermost layer dries out and the dry moss is removed by burning, leaving a large quantity of ash. The sun heats the black and dry surface more readily, so that the mar continues to dry. As a result, the vegetation changes, meadow grass replacing the moss. Summer thawing penetrates deeper, the permafrost recedes, and trees and bushes begin to grow. While these changes are occurring, the fill gradually settles and becomes more stable, and the swellings in the fill disappear. These considerations make obvious the advantages of installing at least elementary facilities for removal of surface water and lowering the ground-water level on mari. Further development of this aspect on the basis of proper experimental investigations will undoubtedly pose and answer the question regarding the feasibility of more extensive and complete drainage of mari prior to construction of the roadbed. Engineer A. Kurtinov finds it advantageous in some cases to apply a supplementary measure for the purpose of accelerating the process of thawing. This measure consists of plowing a drainage belt after some preparatory drying of the surface.

When the fill on a hummocky mar is less than 3 m high, it is necessary to cut the hummocks down. ~~The material obtained in this way may~~

be distributed beneath the fill. In order to drain the upper layer of the bog, it is advisable to dig a drainage ditch at the upgrade side of the fill. This ditch should be located not closer than 10 m from the foot of the berm slope and its depth should not exceed 0.60 m.

In the case of low fills on excessively wet ground, particularly where the ground is absolutely flat, drainage ditches should be constructed on both sides and should have a depth of 2 to 2.5 m. These ditches are located not closer than 10 m from the foot of the fill slope and are designed in the form of braced wooden troughs, as shown in Fig. 115. Special bases are essential in these cases if the fills consist of fine sand, silt, or loam. In the case of a fill exceeding 2 m in height, the special base consists of a 1-m layer of coarse sand, gravel, rubble, or stone (Fig. 116).

In the case of fills 5 m high or higher, the lower portion consisting of well-draining material should extend beyond the slopes in the form of berms. The dimensions of the berms depend on the height of the fill and are presented in Table VIII. These berms are covered with local material to a height depending on the height of the fill (Table VIII). The thickness of the draining layer of the special base should be 1 m, since this is the minimal thickness which would prevent capillary rise of the water and silting of this layer. The tabulated berm dimensions have been established on the basis of the necessity to assure stability of the fill in accordance with its height and to prevent formation of steep slopes of the permafrost hump which may occur in the fill.

TABLE VIII

BERM DIMENSIONS FOR FILLS CONSTRUCTED BY THE PASSIVE METHOD

Height of Fill H in Meters	Top Width of Berm b in Meters	Height of Berm h in Meters
5.0 to 7.5	1.5	1.00
7.5 to 10.0	2.5	1.25
10.0 to 15.0	3.0	1.50
15.0 to 20.0	4.0	2.00

It is advisable to cover the berms with 15 to 30 cm of peat or other insulating material in order to produce flattening of the upper

permafrost limit beneath and beyond the fill. The layer of moss or peat should not be left exposed because it may catch fire when dried by the summer sun. Therefore, this layer should be covered with some local ground. The berm dimensions cannot be determined analytically. Therefore, they are evaluated in accordance with the probable position of the upper permafrost limit. Berms at the base of the fill are not required if there is no permafrost beneath the fill.

It is advantageous to provide ditches on both sides of a fill when feasible. These ditches should be nearly identical in width and particularly in depth. Such ditches should be avoided, or they should be located not closer than 10 m from the fill if it desired to conserve the permafrost beneath the fill. In the absence of permafrost, the ditches should be located in accordance with the usual considerations. It is essential to drain the water from these ditches. Accordingly, the recommended minimum bed slope is 0.003. Use of ditches should be avoided in areas where the roadbed passes over imbedded ice. The requirement that ditches be dug on both sides of the fill is based on the fact that installation of a single ditch causes the roadbed to tilt, so that the rail near the ditch settles more than the other rail. This occurrence has been observed on the Amur Railroad, and is due to thawing of the permafrost on the side nearest the ditch. For the same reason, it is recommended not to dig ditches over wedges of imbedded ice or generally in areas where such ice wedges occur. Similarly, this reason motivates the requirement that the water should be thoroughly drained from the ditches. Since water contains a large quantity of heat, it would cause deep thawing of the permafrost when accumulated in the ditches. This thawing may seriously affect the stability of the fill.

At the conference on Provisional Engineering Standards, conducted by the Academy of Sciences, engineer A. Kurtinov presented extremely interesting and well-designed types of fill for various conditions. These designs were prepared in accordance with the criteria contained in the draft plan for Provisional Engineering Standards originated in 1939 by NIIPS. NKPS 21 . They contain much original material.

Figure 117 shows a fill 2 to 3 m high consisting of well-draining

material. The fill is located on a mar with peat cover, and is constructed in accordance with the principle of permafrost conservation. If the base ground is unreliable, a layer of wood is laid on the ground beneath the fill, the thickness of the layer being sufficient to compensate for the thickness of the peat which has been removed. Peat berms are arranged at the sides of the fill in order to prevent thawing of the permafrost. To prevent fire hazard, the peat berms are covered with 0.15 m of local ground. The peat layer extends along part of the fill slope. If such a fill consists of fine sand, loamy sand, sandy loam, and silt, it is recommended to construct a base of well-draining material of such thickness that this base would extend 0.5 m above the surface of the mar after settling has occurred.

Figure 118 shows a fill consisting of silty material, 2 to 5 m high, erected on a hummocky bog containing mud polygons. The lower portion of the fill consists of a 0.5 m layer of well-draining material. The peat cover is not laid on the slopes. The lower half has a flatter slope than the upper half because this type of material is unstable in the case of steep slopes. The recommended ditches and their dimensions are shown in the drawing. According to this typical design, the hummocks and mounds in the base of such a fill are cut down and removed if the fill is up to 3 m high, while they are cut down and graded only in the berms if the fill exceeds 3 m in height. A 0.5-m layer of peat, covered by a 0.15-m layer of locally excavated ground, is laid on the graded surface of the berm.

Figure 119 shows a design for a high fill (over 5 m) of fine sand and loam, built on a mar covered with peat, moss and grass. As in the previous instances, a base of well-draining material extends 0.50 m above the surface of the mar. To prevent formation of steep slopes of the permafrost hump which penetrates into the fill in the case of high fills, the fill slopes are covered with an insulating layer of peat with a topping of locally excavated material. Peat berms extend along the sides of the fill. The height h and width b of these berms are given in Table IX.

Figure 120 shows a similar fill located on a mar covered with

hummocks and mounds. The design is similar to that shown in Fig. 119, except that the slopes are shallower because silty materials are relatively unstable. Peat is recommended for covering the slopes and berms. The dimensions of the berms are determined in accordance with Table IX. The dimensions and shape of the draining ditches are similar to those shown in Fig. 113.

TABLE IX

DIMENSIONS OF PEAT BERMS

Height of Fill H in Meters	Top Width of Berm b in Meters	Height of Berm h in Meters
5.0 to 7.5	2.00	0.75
7.5 to 10.0	3.00	1.00
10.0 to 15.0	4.00	1.50
15.0 to 20.0	5.00	2.00

The following can be stated with reference to these fill designs. All types of earth fills require a special base or an anticapillary cushion which would prevent saturation of the fill with water. The height of this cushion is 0.50 m. It should be noted that this height is inadequate because a fill of fine-textured material placed on top of a well-draining base will reduce the effective height of the cushion. A considerable part of this height, if not the entire height, will lose its effectiveness because the cushion will tend to become silted due to sifting of fine particles from above. On the other hand, the ground beneath a high fill remains thawed during a considerable period after its construction and contains circulating moisture part of which is transmitted to the fresh fill, along with particles of silt and earth. Thus, the layer of well-draining material tends to become silted also from below. Consequently, an anticapillary cushion 0.5 m thick is hardly adequate; it should be at least 1 m thick. It is difficult to conjecture about the advantage of wooden planks beneath the fill because no practical information is available. However, it is logical to presume that a layer of wood is a poor insulator because it readily becomes saturated with water. Therefore, such a layer is not practical.

The fill profile shown in Fig. 121 was recommended by engineer

E. I. Sukhodolsky for use in a region in which the ground contains an average of 72 to 73 per cent silt. This design specifies preconstruction drainage of the base. This region has peculiar properties. The mean annual temperature ranges between -8° C and -10° C. The minimal winter temperature is about -50° C. Atmospheric precipitation amounts to only 250 to 350 mm per year. There is little snow, but wind velocities of 30 to 40 m per sec often cause snow drifts several meters thick under suitable topographic conditions. Permafrost occurs at shallow depth, while its temperature is -1° C at a depth of 1 m and -5° C to -6° C at a depth of 4 to 5 m. The ground consists primarily of supersaturated fine-textures material containing 72 to 73 per cent silt. Other types of ground, particularly coarse materials, occur rarely. Only three deposits of such material were found along a road stretch of 113 km. The existing fills in this region frequently are extensively deformed due primarily to soaking and sliding of their sides.

Figure 121 shows that the fill slopes are covered with insulating material almost to the very top. The slope steepness is 1:2. The dashed line indicates the probable position of the upper permafrost limit which penetrated into the core of the fill. It should be noted that a special base would have been essential in this fill, despite the relative unavailability of well-draining material along the route of the road. The required material should have been hauled from far away, while careful prospecting for such material should have been undertaken at some distance from the road. Use of coarse material would have reduced the volume of the fills and improved their stability. On the other hand, earth berms are advantageous under these conditions These berms increase the stability of the fill.

2. Cuts

Cuts greatly disturb the permafrost regime. It is advisable to avoid cuts in wet permafrost consisting of earth materials, since the slopes of such cuts tend to slide and the bottom is subject to swelling and settling. In addition, construction conditions become increasingly complicated, so that the cost of construction becomes excessive. It is

particularly recommended to avoid cuts in fine sand, in excessively wet loamy sand, and sandy loam containing silt, silty and slud-like ground.

Since cuts cannot be avoided, procedures are recommended here for stabilizing cuts in various types of permafrost. The design of cuts in well-draining permafrozen ground is an ~~acceptable practice, neverthe~~less it requires considerable care when the ground is wet because the slopes of the cut may be subject to extensive sliding when the ground thaws rapidly during construction operations in the summer. To avoid such sliding, the slopes of cuts during excavation should be covered with 25 to 30 cm of loose peat or 20 to 35 cm of moss. Moreover, this moss or peat cover should be applied simultaneously with exposure of the permafrost layer.

Cuts in permafrost consisting of fine-textured material are relatively inadvisable because they deform more readily. Experience in construction and maintenance of cuts in such ground has demonstrated that, unless costly and complex measures are used, the slopes of such cuts would remain stable only if their steepness does not exceed 1:2 (Fig. 122). The normal width of cuts in ground of this type should be increased by 1 m on each side of the roadbed so as to provide shoulders beyond the drainage ditches; these shoulders would accommodate the thawed ground sliding down the slopes. Prior to thawing and drying of the ground in the slopes, it is advantageous to install temporary drainage troughs, similar to those shown in Fig. 115, in place of the permanent ditches. It is advisable to retard thawing of the slopes during excavation operations. This is achieved by covering the slopes with moss or peat, as indicated previously.

It is essential to avoid cuts in excessively wet permafrost consisting of slud-like and silty material even if it necessitates altering the route or profile of the road. In the exceptional cases when it is impossible to relocate the route, it is recommended to construct such cuts in accordance with special designs and the following specifications. Experience in construction of roadbeds in sliding cuts has demonstrated that the steepness of the slopes should not exceed 1:3. This steepness must be regarded as a minimum and it is advisable not to attempt using any steeper slopes because the final slope will actually be even shallower

than 1:3 if an attempt is made initially to obtain steeper slopes. Cuts in ground of this type should be aligned at a grade of not less than 0.002. If this is not feasible, however, rapid runoff can be obtained if the drainage ditches are designed with a corresponding grade. However, this procedure involves considerable deepening of the ditches and an increased volume of excavation.

It is highly desirable to build berms 2 m wide, outside the ditches on both sides of the roadbed. These berms constitute additional protection for the ditches and the road against possible slides (Fig. 123). In addition, it is advisable to remove a layer of ground 2.0 to 2.5 m thick from the bottom of the cut and to replace it with slag, stone, rubble, or gravel; coarse sand may be used if necessary. This procedure is recommended on the basis of experimental investigation and in accordance with opinions of persons with corresponding construction experience.

If furnace clinkers are available in the vicinity of the cut, it is advisable to deposit at least 0.6 m of the clinkers at the top of the backfill in the pit formed when the original ground was removed from the bottom of the cut. As stated previously, the ditches should be designed in the form of deep drainage troughs (Fig. 115). In order to intensify the drying of the ground, it is advisable to cut longitudinal surface drains along the slopes of the cut. These drains should be 0.60 m wide, and 0.60 m deep, filled with fascines or with boulders laid on a layer of moss or peat if fascines are not available.

The surface of the slopes should be covered with a 0.30-m layer of loose peat or moss during excavation of the cut. The function of this cover is to retard thawing of the ground in the slopes. This cover, in conjunction with the longitudinal drains along the slopes, should facilitate more intense drying of the ground and decrease its tendency to slide. The drains should consist primarily of fascines, and the use of rock is permissible only if fascines are not available. The rock should be surrounded by a layer of peat or moss in order to prevent its becoming heated by the sun or the air and the subsequent transfer of heat into the ground.

A retaining wall of fascines or brushwood should be installed at

the foot of the slope in cuts exceeding 2 m in depth. The function of this wall is to detain the peat or moss layer on the slopes and, to some extent, the sliding ground. It is recommended to carry out excavation of cuts during the winter.

Cuts in earth material should be bounded by ditches along the slopes. In the case of rocky slopes, these ditches are installed if the slope is not steeper than 1:1. Use of deep ditches (deeper than 0.60 m) should be avoided in order to prevent excessive disturbance of the permafrost. To assure rapid runoff, the longitudinal grade of the ditch should be not less than 0.003. The required cross section of the ditch should involve extra width instead of depth; therefore, the cross section shown in Fig. 124 is recommended. The distance between the edge of the cut sidewall and the edge of the ditch sidewall should be not greater than 20 m in the case of earth material and not greater than 5 m in the case of rock or rocky ground.

Figure 125 shows the design of a cut in excessively wet permafrost consisting of silty material. This design was prepared by engineer A. Kurtinov in accordance with the 1939 draft plan mentioned previously. However, this design has original aspects. The slopes of the cut a covered with peat or moss in order to retard and reduce thawing. The steepness of the slopes is 1:3. The bottom of the cut is covered with a layer of wooden planks. The thickness of this layer is to be determined analytically. The drainage ditches are designed in the form of deep troughs. It should be noted that the layer of wooden planks is hardly of any value, as it might readily become saturated with water and would not act as an insulator.

Figure 126 shows a cut with banks, made in swelling ground. This design specifies removal of the ground at the bottom of the cut to the depth of seasonal freezing and backfilling with nonswelling ground. The troughs are designed in accordance with Fig. 115. It should be noted that the slopes in such a cut should be covered as shown in Fig. 125, since the swelling ground, presumably clay loam or silt, would tend to slide because of intensive thawing of the permafrost in the slopes.

Figure 127 shows Sukhodolsky's design of a cut in silty, slud-like ground of low bearing capacity. In addition to the regular insulation layer on the slopes of the cut, there is a double layer of insulation consisting of wood and peat beneath the roadbed ballast. Rock or rubble is recommended as ballast. The ditches are replaced by wooden troughs. Seasonal insulation of moss or peat, about 0.3 m thick, is placed in the trough during midsummer, approximately during the second half of July. This insulation should be removed at the beginning of winter in order to facilitate intense freezing of the ground adjacent to the troughs. The ditches should be kept free of snow.

At the beginning of the warm season, when the lower layer of the troughs thaws and begins to function, use of insulation in the trough, in the opinion of Sukhodolsky, would prevent further excessive warming of the lower layer, while not interfering with its functioning as a drain. Accordingly, the seasonal insulation in the troughs is effective to a certain extent as a continuous heat insulator along the entire profile of the cut. Analytical calculations indicate that the thickness of the peat layer should be 0.50 m under given local conditions. The peat layer together with the wooden flooring provides a safety factor of 1.25. It is inadvisable to use a peat layer thicker than 0.40 m because of its elasticity. The thickness of the layer of well-draining material should not be increased arbitrarily because an excessively thick layer would greatly impair drainage.

Sukhodolsky acknowledges that cuts of this type would be extremely costly, but he notes that this design is justified because of the exceptionally unfavorable conditions under which the roadbed in the cuts would have to be maintained in the given region and because of the relatively short length of such cuts involved in the case of roads planned under similar conditions.

It is doubtful whether elastic materials such as moss or peat are suitable for use in railway roadbeds. The roadbed would probably be unsteady.

E. Measures Against Icings

Control of icings often is complex and difficult. In some cases

the measures against icings are relatively ineffective. Therefore, when selecting a construction site or laying out a road, it is advisable to avoid areas where icings occur. Since this is not always feasible because of engineering and economic considerations and since construction of a building or, particularly, a road may create conditions leading to formation of icings in places where they have not existed previously, it is essential to take measures to protect the structures or roads against icings.

If a given structure--whether a building, bridge, or embankment--is endangered by an icing, it is advisable to use the active method of counteracting the icing phenomena. This method essentially consists of the following two major measures: (1) drainage of the locality and (2) construction of frost belts. A. M. Chekotillo [22] recommends the following two additional measures in the case of bridges: (1) deepening and straightening of river channels, and (2) insulation of stream beds.

The passive method comprises efforts to drain the water forming the icing and construction of retaining walls and barriers to prevent spreading of the icing. It should be noted that achievement of rapid and complete effectiveness in control of icings in any given case requires the application of several measures based on both the active and passive methods and combined in accordance with the actual local conditions. The effectiveness of the measures depends upon careful investigation of the causes of the icing phenomena and upon the corresponding course of action. The causes and characteristics of icings, as well as their effect on structures, were discussed in Chapters I and II. As stated there, icings are of the following three types: ground, river, and spring.

The simplest and most effective method of controlling ground icings consists of careful drainage of the area by means of surface ditches or under suitable topographic and hydrogeological conditions, by means of a system of subsurface drains which collect the water occurring in a given layer as well as any other ground water and divert it from the structure. However, construction of surface drainage ditches requires definite care and detailed consideration of local conditions. Drainage ditches may cause deep freezing of the ground and formation of icings nearby. In addition,

drainage operations lower the upper permafrost limit, which is not always desirable. This is not advantageous, for example, when existing structures in this area are based on the permafrost and are designed in accordance with the principle of conservation of the permafrost layer.

Ditches require maintenance consisting of periodic cleaning and repair. Their dimensions and design are determined in accordance with local conditions and generally do not differ from the ordinary. If the drainage operations are not feasible or cannot be carried out on a scale sufficient to eliminate completely the icings, due to particular local conditions, it is necessary to use the method of the so-called frost belt.

Frost belts are arranged in the form of a wide ditch or a ditch with an adjacent strip of ground (wing) from which the surface cover has been removed, as shown in Fig. 128. A frost belt may also consist of a single strip of ground from which the surface cover has been removed, as shown in Fig. 129. Frost belts are devices which facilitate rapid and deep thawing of the active layer in definite places, so that the movement of ground water is blocked and, consequently, the icing is prevented from reaching the roadbed, the bridge, the culvert, the building, or any other structure. Freezing proceeds more rapidly under the bare surface of the ground and beneath the bottom of the ditch because the cold penetrates there more readily and rapidly. The layer of seasonal freezing merges with the permafrost or another impervious layer and forms a barrier so that the icing forms near the ditch, that is, the frost belt produces the desired effect. If the icing is large, a number of frost belts are arranged at regular intervals so as to cause gradual disappearance of the icing.

A frost belt for control of ground icings should be designed in each case in accordance with the character and dimensions of the icing as well as the entire complex of local conditions. Accordingly, it is difficult to specify exact dimensions for width, depth, length, and distance from the roadbed or structure. Nevertheless, existing experience in design of frost belts makes it possible to recommend the following approximate dimensions:

(1) Width of frost belt in the form of a ditch--5 to 10 m.

(2) Depth of frost belt in the form of a ditch--0.5 to 1.0 m.

(3) Width of stripped belt (wing)--10 to 15 m.

(4) Distance from structure--50 to 100 m.

It is obvious that use of a frost belt is feasible only if an impervious layer occurs at some distance below the ground surface. In order to construct a frost belt, it is necessary to determine the direction of flow of the ground water because the frost belt is effective only if it is located approximately normal to the flow of the ground water.

The construction of frost belts on the Amur-Yakutsk Road is an interesting example of protection against ground icings [7]. Icings recurred near a number of homes. A drainage ditch 257 m long was dug 70 m from the buildings in order to protect the buildings from the icings and to divert the water flowing down steep hillsides nearby. The ditch was 5 m wide at the top, 0.8 m wide at the bottom, and 2 m deep. The icing which formed during the following year did not reach the houses (Fig. 130). With regard to this particular case, it should be noted that the belt was constructed too close to the buildings. V. G. Petrov studied these icings and recommended construction of a second, smaller frost belt. Figure 131 shows Petrov's design of a frost belt which may be regarded as typical for ground icings. The ditch is 5 m wide and 1 m deep, while the wing is 10 m wide on the uphill side. In the given instance the belt was 290 m long and was located 40 to 50 m from the axis of the road. The icing which had formerly formed on the road, shown by shading in Fig. 131 moved uphill beyond the belt and did not affect the road again.

Frost belts against ground icings require constant inspection. Their effective period, usually not longer than a few years, may be extended if they are covered during the summer with insulation layers such as peat or moss. The advantage of this procedure has been well established in practice and is explained by the fact that the depth of summer thawing of the ground beneath the belt is greater than the depth of winter freezing. As a result, the water percolates through the thawed layer and there is nothing to prevent ready formation of icings at likely places.

Attempts have been made to avoid deep summer thawing of the frost belt by covering the belt for the summer with an insulating moss layer 30 cm thick. The results were highly satisfactory. The belts were effective. It is obvious that the moss cover should be removed at the beginning of autumn and should be stacked away from the belt. The effectiveness of the belts is improved when the winter snow is removed from them. Snow on a frost belt greatly reduces the depth and rate of freezing of the ground beneath the belt.

In addition to their primary function of keeping icings away from structures by retaining the ground water at some distance from the structures, frost belts may be used to reduce swelling of the ground at bridge piers and structural foundations.

When river icings are sufficiently large to constitute a hazard to the bridge or other road structure and to clog the bridge or the culvert with ice, frost belts are constructed in order to cause the icing to form upstream at a safe distance from the structure. These belts are constructed in the form of ditches in the ice across the river. These ditches are 1 to 3 m deep and 3 to 5 m wide, as shown in Fig. 132. They extend some distance into the river banks and facilitate more rapid and deeper freezing of the underlying ground in the river bed and in the banks. Thus, the water flowing in the conglomerate bed and banks is blocked by an ice barrier at a certain distance from the road structure. This barrier forces the water to the surface and causes formation of an icing at the designated place.

The distance between the road structure and the frost belt depends on the size of the icing and the contour of the river bed. This distance should be approximately 100 to 300 m. It is advantageous to construct the belts on sandbanks in order to reduce the volume of excavation and to accelerate the formation of the ice barrier. The method of "depletion," that is, a succession of frost belts, is effective in the case of very large icings.

In the case of small river icings and if snow is available, it is recommended to use a control method consisting of wide snow dikes across the

river. These dikes are erected under the bridge and on the nearest upstream sandbank. They reduce freezing of the river. Several holes are made in the ice downstream of the bridge in order to allow the water to emerge on the surface of the ice in case the flow is blocked by frozen sandbanks on the downstream side.

The frost belts on the Yakokit River (Fig. 133) constitute a most interesting example of the application of this method for the protection of structures. Their design is described by V. G. Petrov in his aforementioned book on icings. An excerpt follows.

> "To protect a bridge under construction across the Yakokit River from an unexpected excess of water, it was necessary to construct protective belts consisting of dikes made of snow and ice. Four belts were constructed. The first three belts were so arranged as to deflect the water into adjacent minor channels. The fourth belt is located 10 m upstream of the old bridge and consists of a trench excavated in the ice down to the river bed (by the refrigeration method) in order to form a frozen plug blocking the water upstream of the bridge under construction. This belt is constructed as a safeguard against possible catastrophe in case the first three belts are unable to resist the pressure of the water. The dimensions of the belts are as follows. The first (of snow) is 25 m long and is located 260 m from the old bridge. The second (of snow) is 25 m long and is located 160 m from the old bridge. The third (of spoil, that is, of snow and ice) is 50 m long and is located 60 m from the old bridge. The fourth (a ditch in the river ice excavated to the river bed) is 50 m long and is located 10 m from the bridge. Accordingly, the intervals between the belts are respectively 200, 100, and 50 m."

Spring icings are controlled by capping the spring and diverting its flow to a suitable place by means of subsurface drains or a pipe laid on the ground surface and protected against freezing. M. Ya. Chernishev recommends capping of springs by means of a simple underground timber gallery, as shown in Fig. 134. These galleries differ from the ordinary ones in that they are well insulated with an earth fill and a layer of moss or peat. This layer should be 0.75 to 1.0 m thick and topped by local earth material as protection against fire hazard. Ordinary wells may be used for capping of springs.

The objective of insulating a stream bed is to assure the flow of water. A. M. Chekotillo recommends the arrangement shown in Fig. 135. The

channel is covered with logs and poles arranged longitudinally and transversely. They rest on the banks and on sawhorses placed at midstream. The poles are covered with brushwood and twigs which are topped with a layer of moss or peat 20 to 30 cm thick. If snow is available, the brushwood may be covered with a snow layer of about 50 cm.

An icing or an icing mound near a structure may be eliminated by means of small charges of thermite. The size of the charge is determined at the site in accordance with the design of the bridge and its supports and with the distance of the charge from the bridge. Explosives may be used if the explosions do not constitute any hazard to the structure. It should be noted that the use of thermite is a suitable method for eliminating ice jams in rivers.

CHAPTER V

SPECIAL CONSTRUCTION PROCEDURES UNDER PERMAFROST CONDITIONS

A. Procedures for Construction of Structural Foundations and Supports

1. General Remarks

The stability of structures under permafrost conditions depends largely on carrying out the construction operations in full accord with the local permafrost characteristics. Accordingly, it is important that the men in charge of construction operations should be familiar with permafrost and its phenomena. It is equally important to observe all special requirements and specifications involved. In addition, the construction engineer should make a thorough study of the area. He should analyze the results of investigations and surveys and make a personal inspection of the entire construction area. The operations should be accurately and consistently coordinated with the construction method specified in the plan. The area involved usually constitutes virtually virgin Siberian forest, so that it is necessary to begin with preparatory work comprising primarily felling of trees, some drainage operations, and construction of temporary roads.

If construction is based on the principle of conserving the permafrost, it is undesirable to clear the forest or remove the scrub and surface

covers except on the areas designated as prospective sites for the roadbed, buildings, and road structures. It is particularly important to preserve all vegetation at the following places: (1) on slopes of steep hills where snow slides and rock slides may occur, (2) on slopes of hills which tend to slide and on slopes where permafrost occurs at shallow depth, (3) near fills extending 3 m in height and cuts exceeding 2 m in depth, and (4) near the south sides of buildings.

In regions of deep winter freezing, where permafrost is absent or sporadic, and in regions where the permafrost occurs at great depth or undergoes intensive degradation, it is essential to carry out extensive removal of scrub and vegetation cover in order to facilitate drainage. The surface cover is eliminated by burning the grass and other vegetation. Forests should be preserved whenever possible, but bushes, scrub, and deadwood should be cleared away. Preservation of forests prevents any rise in ground-water level and formation of swamps. This is particularly important in permafrost regions where the relative dryness of the active layer is a major factor in the stability of the structures and the cost of construction.

Timely drainage is essential and should be applied on a large scale within the entire construction area, except in cases when it is required to conserve the existing permafrost regime. General preliminary meliorative operations are undertaken in all cases of construction in regions of deep freezing where the permafrost undergoes degradation, or in areas where it is proposed to eliminate the permafrost, as well as on sites of prospective cuts.

Drainage may be accomplished by means of open ditches arranged in accordance with local conditions. To avoid formation of swamps in the construction areas, it is particularly essential to divert the water into an active stream. During melioration operations and particularly during removal of vegetation, it is essential to bear in mind that brushwood facilitates accumulation of water because it retards runoff and prevents effective aeration of the ground surface, while large trees, in contrast, retain a considerable proportion of atmospheric precipitation in their roots and

absorb a large quantity of moisture from the ground during the growing season, so that they facilitate drying of the area.

Elimination of moss, peat, and grass covers is carried out in the autumn by means of burning. Construction of temporary roads in regions where permafrost is absent or occurs at great depth, as well as in areas where the permafrost is in the process of degradation, is carried out in the usual manner, swampy areas being by-passed whenever possible. In regions of stable permafrost occurring immediately beneath the active layer, the temporary road should be constructed as far from the structure as possible in order to avoid excessive disturbance of the permafrost regime. In swampy areas, a temporary road should be built of beams laid on longitudinal logs and covered with ground. Brushwood causeways may be used. If the road is scheduled to accommodate tractors, a double row of beams should be laid on ties spaced at intervals of 1 m, or a flooring of timbers may be used in place of the beams. The upper layer should be covered with well-draining ground material. Figure 136 shows a temporary road over a marshy stretch along the Amur Railroad. This temporary road was constructed as described above.

All auxiliary and temporary structures should be located as far from the major structures as possible. During construction of small bridges, in particular, all auxiliary structures both on land and in the water should be located 30 to 50 m downstream. Restricted zones should be established in areas where economic activities of other organizations may cause disturbance of the regime of the locality or otherwise interfere with construction and operation of the structures. The sizes of such zones are determined in accordance with local conditions. Restricted zones may have to be established in areas where it might be necessary to erect special facilities or to carry out special operations at some distance from the major construction site in order to conserve the existing permafrost regime or to assure structural stability. In addition, such zones may be necessary in areas where erected of new structures might interfere with normal existence and functioning of the major structure. The areas which must be included in the restricted zone comprise, for example, sites

where large ground or river icings may form or places where imbedded ice occurs.

2. Special Instructions Regarding Construction of Building Foundations

In the case of building foundations, the construction operations depend on the construction method involved, that is, active or passive method. Winter construction is recommended in the case of the passive method as well as when the ground is excessively wet and would require extensive pumping. In the case of the passive method, construction operations during the winter would disturb the permafrost very little and it would not be difficult to conserve the permafrost at the base of the structure. A foundation pit excavated in the winter would not be affected by atmospheric heat. Only little thawed ground, formed during backfilling of the foundation pit, would remain in the ground among the permafrost. In addition, the temperature of a foundation erected in the winter would be lower than that of a foundation erected in the summer. It is obvious that the heat stored in the backfill and the foundation during the winter will be considerably less than the heat stored during the summer. In addition, the permafrost surface and the walls of the pit extended into the permafrost are subjected to considerably lesser heating.

On the other hand, the design of some structures may specify imbedding the lower portion of the foundations in the permafrost layer in order to prevent swelling of the active layer (Fig. 76). In such a case, winter construction of the supports is not only desirable but imperative because it is necessary to adfreeze the completed support to the ground. This can be accomplished only by winter refreezing of the ground excavated from the pit. Otherwise, it would be necessary to apply artificial freezing or assume that the support may heave due to swelling of the active layer.

Summer construction of the foundation and backfilling of the pit with thawed ground is not permissible because winter freezing and swelling of the active layer will occur prior to complete freezing of the backfill. Similarly, leaving the pit open during the summer and backfilling it in the winter is not permissible because the permafrost will thaw unless special measures are taken, such as covering the pit with a highly insulating material. Such a procedure is not always feasible or convenient because it

is difficult adequately to insulate a pit against heat and, in addition, the pit has to be waterproofed against both surface and ground water, which is virtually impossible. Therefore, whenever the plan calls for conservation of the permafrost and its utilization to protect the foundations against heaving, construction operations during the cold season of the year are highly recommended.

In the case of excessively wet ground, winter construction operations are advantageous because excavation in the water-bearing strata is readily accomplished by means of freezing, and it is not necessary to use complicated and costly drainage operations. Freezing of ground and even water and streams during construction of foundations is a widely used, simple, and cheap method. The method is described in A-6 of this chapter.

It is quite difficult to excavate frozen ground from foundation pits if the work is carried out in the usual way. Therefore, it is necessary to use special procedures. These methods are discussed later. Only the most rational methods for construction of foundations will be discussed here. These methods comprise excavation by means of explosives, thawing the ground by means of steam points, and thawing by fires. The first two methods are recommended for construction in accordance with the principle of permafrost conservation because these methods involve minimum disturbance of the permafrost, which is an important factor in the passive construction method.

After the pit has been excavated to the design depth, the thinnest possible layer of wet sand is placed on the permafrost surface in order to level it out and a wooden grillage is laid over the sand, covering the entire bottom of the excavation (Fig. 137). The foundation is erected on the grillage. Since the foundation usually is of concrete or rubble concrete, forms are erected over the grillage. These forms may remain in place after the foundation has been constructed. The pit within the permafrost is backfilled with wet sand or sandy ground, and this backfill is allowed to freeze before backfilling the upper portion of the pit.

Lowering of wells and piles into the permafrost layer is achieved most readily by means of the American steam point because thawing of

the ground proceeds rapidly and it is feasible to thaw out not more than the necessary volume of ground.

If it is desired to conserve the permafrost, it is necessary to prevent flow of surface and ground water into the pit. Instructions regarding construction operations within protective enclosures are presented in A-4 of this chapter.

3. Use of Steam Points and Electrical Thawing of the Ground

The American steam point, shown in Fig. 138, is a simple steel steam pipe 20 to 40 mm in diameter and 5 to 8 m long, with a nozzle at one end and a heavy head or block at the other end. The block weighs 10 to 15 kg and is equipped with horizontal handlebars. A flexible hose connects the upper end of the pipe to a steam boiler.

There are two types of nozzles (Fig. 139), for use in soft and firm ground, respectively. The nozzles have a number of apertures through which the steam is ejected. The flexible hose is usually attached by means of an elbow and is equipped with a valve to regulate the steam flow. The steam is supplied by a steam boiler, passes through the pipe which has been lowered into the frozen layer, and thaws the ground. One boiler can supply several points (Fig. 140).

The tip (or nozzle) is lowered into the ground by pressing with the hand on the handlebar of the block or by applying light blows with a mallet or hammer on the block. Simultaneously, steam is released through the nozzle (Fig. 141). It is most convenient to work when standing on saw-horses, while a square plate of wood or sheet iron, 1 m square and having a center aperture to accommodate the point, is laid on the ground.

The steam is delivered at a pressure of 2 to 8 atm so that it is ejected from the nozzle with considerable force. The shield on the ground is necessary in order to protect the workmen from being injured by the steam and the particles of ground. One and sometimes two men can operate the point. In place of the special nozzle shown in Fig. 138, it is possible to draw out the end of the pipe and drill several holes in its sides. Such points can be prepared in the most poorly equipped workshop.

The rate of thawing of the ground is quite high and depends upon the diameter of the nozzle and the steam pressure. The greater the diameter and the higher the pressure, the more intense the thawing of the ground. According to engineer S. Ya. Bozhenkov, the rates of thawing of ordinary frozen ground are as given in Table X.

TABLE X

RATE OF THAWING WITH STEAM INJECTOR

Pit Diameter in Centimeters	Time in Minutes	Volume of Thawed Ground in Cubic Meters
40	25	0.25
50	35	0.40
60	45	0.55
80	60	1.00

These rates depend on the diameter of the thawed pit and were obtained at a steam pressure of about 5 atm, at a thawing depth of about 2 m, and using a 20 mm nozzle. When the steam pressure is increased from 5 to 8 atm, the rate of thawing increases approximately 10 per cent per additional atmosphere, that is, the rate increases 30 per cent when the pressure is increased to 8 atm. According to Professor Shchelokov, thawing of 1 cu m of common frozen ground in the Moscow region cost 2 rubles 34 kopecks in 1938-39.

The record shows that 8 to 9 bores in permafrost containing 60 to 70 per cent moisture, 2.7 m deep and 35 to 40 cm in diameter, were obtained with a steam point 40 mm in diameter during 10 man-hr. Another report discloses that it required only 20 min to thaw out a pile shaft 6 to 7 m deep, using steam pressure of about 8 atm. The concensus of opinion is that the steam boilers should have a capacity of not less than 6 atm, and preferably 8 atm. However, it is feasible to use low pressures of 4 to 5 atm, but the work would proceed more slowly.

It is essential to insulate the steam lines in order to prevent cooling of the steam as it passes from the boiler to the nozzle. The pressure drop due to cooling of the hose is quite considerable when the outside temperature is -30° C to -40° C.

After the ground has been thawed by the steam point, the piles may be lowered in the usual manner. The pile is readily lowered into the

ground; it requires a maximum of 20 min to lower a pile to a depth of 6 to 7 m.

The best time for driving piles with the aid of steam points is at the beginning of winter during the first frosts. Under these conditions, it is unnecessary to thaw the active layer, since it can be excavated to the permafrost, so that lowering of the piles proceeds from the permafrost surface proper. The piles driven into the thawed permafrost remain in the open pit and are not backfilled until spring. The pit within the active layer is filled with rubble immediately prior to the arrival of the warm season. Should it be necessary to reduce the period during which the pile remains uncovered, it is essential to determine whether the permafrost has been restored. This can be done by means of drilling or pulling out the pile. The time required to restore the permafrost thawed by a steam point is indeterminate and depends upon local conditions. According to Bikov, the permafrost is restored during the winter within 1.5 to 2.0 months.

Some engineers are of the opinion that the months of January to June constitute the best season for steaming the ground for pile foundations. The pit in which steaming operations are carried out should be protected against penetration of snow and water. The water should be pumped out at regular intervals and the ice should be chopped away. The pit should be covered with boards in order to keep it free of snow.

The steam point has been widely used in the United States (Alaska); hence it is referred to as "American." In American practice, large areas are simultaneously thawed by steam points. This is accomplished by means of a large number of points arranged along parallel steam lines.

Use of steam points for various types of work was recommended in the preceding. This device actually constitutes one of the simplest and most effective methods for excavation of frozen ground [23, 24].*

An interesting type of thawing operation by means of electricity was used at Solikamsk. This procedure involved placing an electrical heating unit in a 16 mm bore drilled in the frozen ground. The heating unit remained there for 6 hr and thawed a column of frozen ground 0.8 m in diameter. The

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* These publications contain highly interesting and recent data on ground thawing by means of steam and electricity.

power used was only 0.3 to 0.4 kwhr. The heating unit was designed as follows. A round steel bar, 19 mm in diameter, was wrapped in 3 mm of sheet asbestos which was tied spirally with fine furnace or binding wire. This spiral was covered with asbestos and placed in a 50 mm gas pipe. The lower end of the pipe was sealed, the entire unit was lowered into the bore, and the pipe was connected to a source of electric current [23, 24].

4. Specific Instructions Regarding Erection of Bridge Foundations and Other Special Structures

In cases where it is proposed to build supports on the principle of permafrost conservation, the best time for excavation of foundation pits extending into the permafrost is during the first frosts, that is, at the beginning of winter. Construction operations under these conditions have a minor effect on the permafrost regime. Moreover, excavation in the active layer does not present any difficulty because the layer is not yet frozen. If the active layer is wet, the freezing method can be applied. It is advantageous to conduct excavation operations during the cold season also in areas where permafrost is absent or occurs at great depth and the ground is wet, as freezing of the foundation pit is applicable under these conditions.

Excavation of permafrost may utilize hand tools, pneumatic instruments, fire, steam points, and explosives. The last two methods are recommended if it is proposed to conserve the permafrost, since these methods cause the least disturbance of the permafrost regime. The foundation pit should be wider than the projected foundation. The magnitude of the additional width depends upon the nature of the ground; 1.0 to 1.5 m is sufficient in the case of sandy or gravelly ground, while 1.5 to 2.0 m should be used in the case of loamy or silty ground. When fire is used to thaw the ground, the walls of the pit excavated in the permafrost should be covered with moss, peat, straw, or a similar material, and faced with planks in order to avoid thawing and creeping of the walls.

During either summer or winter excavation operations, it is essential adequately to protect the pit from surface or ground water,

using drainage installations or pumping, in order to maintain the frozen state of the foundation base, if necessary, and to avoid general complications. If the excavation was prepared in the autumn or winter, the foundation should be constructed immediately after completion of the excavation operations and the work should proceed within suitable temporary enclosures. If it is desired to conserve the permafrost at the foundation base and the work proceeds within the protective enclosures, it is essential to observe the following instructions. Prior to heating the enclosure, the pit walls within the active layer should be covered with moss, peat, or straw, and faced with planks to avoid heat transfer from the pit to the ground. The air temperature within the pit should be not lower than zero or higher than 5° C. It should be noted here that certain types of cement do not harden or set within this temperature range. Therefore, the temperature in the pit should be regulated accordingly. The composition and quality of the mortar and concrete should be the same as in the case of work at low temperatures. It is essential to heat the gravel, sand, and water for the concrete. Vibration accelerates hardening of the concrete. The stoves for heating the pits should be located above the pit at ground level, otherwise the temperature in the pit will rise above the allowable limit and will affect the permafrost. The stoves may be housed in the structure used for heating the aggregate.

After the base layer and timber grillage are laid on the surface of the exposed permafrost, the spaces between the grillage and the pit walls should be filled with thoroughly tamped wet sand. The previously prepared forms for the concrete are placed then on the grillage and concrete is poured to a height of 0.5 to 1.0 m above ground level. Pouring the concrete or laying the masonry should proceed as rapidly as possible in order to minimize heating of the permafrost. This partially completed foundation is kept in the heated enclosure sufficiently long to allow the mortar or concrete to attain the required strength, after which the enclosure is cooled and dismantled. At the same time, the lower portion of the pit wall insulation (moss, peat, planks) is removed and the spaces between the foundation and the pit walls within the permafrost layer are filled with wet sand. This fill is subjected to frost action in order to restore the permafrost.

If the construction schedule permits, it is advisable to cover the pit with planks and leave it this way until spring.

After the sand fill within the permafrost layer and the thawed pit walls have been frozen, the remainder of the foundation pit is filled at the end of winter with rubble or other well-draining material if this involves no complications and is not costly. The wall covers within the active layer should remain in place as protection against silting of the fill. Concrete forms, if any, should remain in place also. The foundation sides within the active layer should be given a smooth finish as construction work progresses.

The suprasurface part of the support may be erected during the summer. If it is necessary to continue the work during the winter in a heated enclosure, it is imperative to place an insulating layer around the support on top of the pit which has been previously backfilled to ground level. This layer should consist of slag, ash, moss, peat, straw, pine needles, or a similar material, and should be covered with ground. The layer should be at least 50 cm thick. It is advantageous to provide special channels beneath this insulating layer in order to facilitate circulation of cold air to the footing. If it is proposed to conserve the permafrost, backfilling of abutment excavations should be carried out before the warm season sets in. The backfill material should be relatively coarse, preferably frozen, and should be well tamped. If well-draining material is not available and other material is used, drainage layers of gravel or rubble should be installed at the rear of the abutment.

If a rock base is available, supports resting on cribs may be erected during any season of the year. If the rock fill beneath the crib rests on permafrost, it is advisable to proceed with construction during the winter. In this case, the lower logs of the crib are installed during the spring and the pit is completely backfilled. If the active layer is water bearing, it is more convenient to carry out construction during the winter and use the freezing method even if a rock base is available. The crib should be heavy; accordingly, it is advisable to fill the crib with material of assorted sizes: cobble-stones, rubble, gravel, or coarse sand.

All auxiliary structures, material dumps, temporary roads, and workers' quarters should be located downstream of the bridge under construction, at a distance of at least 25 m from the bridge axis. It is imperative not to disturb the regime of the area upstream of the bridge.

5. Special Instructions Regarding Construction of Subsurface Structures

When tunnels are faced with masonry or concrete, the section of the tunnel in which this operation takes place should be maintained at a temperature sufficient to assure setting and hardening of the concrete (not lower than 5^{o} C). This temperature should be maintained sufficiently long to allow the mortar or concrete to develop its design strength. Use of high-grade cement and accelerated rate of work would shorten the required time interval.

If the frozen ground in the tunnel may become slud-like when thawed, or if it consists of individual chunks of rock cemented by interstitial ice, it is necessary to use good insulation during construction of the foundations, walls, and arch in order to preserve the frozen state of the ground while the mortar or concrete is in the process of hardening.

Waterproofing is an aspect requiring particular care. Waterproofing operations must be thorough. It is essential to bear in mind that ground water often is highly aggressive in many cases under these conditions and may damage the masonry if proper preventive measures are not taken.

6. Freezing of Foundation Pits and Their Protection Against Inflow of Water

Excavation operations in permafrost areas often encounter almost insuperable difficulty due to excessive ground water and the impossibility of using sheet piling. In addition, if the plans specify conservation of the permafrost at the foundation base, special measures are necessary to protect the excavation against inflow of surface or ground water.

Removal of water from an excavation is usually a costly operation because the excessive moisture of the ground and the flow of ground water necessitate the use of large centrifugal pumps with mechanical drives. On

the other hand, if the work is done during the winter, pumping is avoided and the operations are simpler and cheaper because it is feasible to apply the method of freezing of the ground using natural cold. This method is applicable even in the southern regions of permafrost where the air temperature in the winter reaches -30° to -40° C. The advantage of the freezing method is due to the fact that this method causes only negligible disturbance of the permafrost regime. In addition, this method is simple and cheap [25].

It is feasible to freeze not only the ground but also flowing water in a river as well. As a result, freezing may be substituted for the complex and expensive method of caissons when piers are erected in water. Experience in the use of the freezing method is quite extensive and this method has been sufficiently tested in practice. Figure 142 illustrates an interesting example of application of the freezing method to construction of supports. It shows the pier of a bridge with 85 m spans, extending 12.2 m below the river bed or 13.9 m below the water surface. Freezing was applied to a water layer 1.7 m thick, a layer of pebble 2.5 m thick, and a layer of brown silty loam 9.7 m thick.

Freezing of an excavation in the ground is carried out as follows (Fig. 143). When the frosts set in, the excavation operations proceed until the ground-water level is reached, within the layer where work can be carried out without pumping. The pit is excavated in such a way that longitudinal and transverse earth partitions 30 to 50 cm thick remain in place, so that several compartments are formed, each measuring at least 2 by 2 m, as shown in Fig. 144. This compartmentation is necessary as a safeguard against flooding of the pit during subsequent excavation operations. Should that occur, it would be necessary either to pump out large quantities of water or, if the water is permitted to freeze, to remove large masses of ice. When partitions are used, however, only one compartment may become flooded and the situation can be readily remedied.

Freezing of the pit sidewalls is accelerated if the vegetation or moss cover is stripped from the ground surface around the pit. The

excavated pit is exposed to the cold air for a few days until the bottom of the pit freezes to a depth of 30 to 50 cm. This requires four to seven days, depending on the temperature of the air. To facilitate freezing, the snow should be removed from the bottom of the pit and the exposed surface around the pit. The depth of freezing is determined by drilling the frozen ground at several places. Part of this frozen layer is then removed, the excavation proceeding until the remaining layer is at least 20 cm thick. Afterwards, the pit is permitted to freeze again, and a portion of the newly frozen layer is excavated subsequently. This process is repeated until the desired depth is reached.

Picks and crowbars are used during excavation, but it may be feasible to thaw the ground by means of bonfires or to use explosives. The latter method requires experience, skill and caution. When bonfires are used, the walls of the excavation should be covered with moss, peat, or straw and enclosed in planks in order to prevent heat transfer to the walls.

If water emerges into the pit through the drill hole which was used in determining the depth of freezing, the hole should be closed with a wooden or iron plug. If a spring occurs in the pit, it should be diverted or capped. Freezing is accelerated if iron crowbars or pipes are driven into the ground. The upper end of the pipe should remain open, while the lower end should be sealed.

Horse-driven ventilators were used on the Trans-Baikal Railroad in order to accelerate freezing of excavations 3.5 to 4.0 m deep. The ventilators blew cold air on the bottoms of the excavations. Professor A. N. Passek recommends utilizing the wind for this purpose by installing propellers or special funnels to direct the air into the excavation [26]. After the pit has been excavated to the required depth, its walls are faced with planks underlain by moss, straw, or other insulating material. A heated enclosure is installed then, within which the work proceeds as usual. A-4 of this chapter contains the pertinent information.

Freezing of water in a river, and subsequently of the river bed, is conducted in an analogous manner. In order to accelerate the freezing operation, it is recommended to retard the flow at the area involved by

driving sheet piling or ordinary stakes, as shown in Fig. 145. The excavation is divided into compartments, as in the previous cases, and the ice is chopped away as it forms. It is necessary to take into consideration the fact that water freezes approximately three times slower than ground. Professor Passek, who had extensive experience in operations of this type, recommends the use of pipes to accelerate freezing of the water. Each pipe is lowered into the water with one end closed or bent in the form of a U so as to permit maximum free circulation of the air (Fig. 146). This method accelerates freezing one-and-one-half to two times. Pipes 6 to 15 cm in diameter are recommended.

According to Passek, application of iron tanks is most advantageous (Fig. 147). These tanks are prepared by welding or riveting sheet iron 3 to 5 mm thick into cylinders of minimum diameter 45 cm. The tanks are waterproofed. The bottom of the tank is located above the lower end of the cylinder, so that the tank can be sunk 20 to 40 cm into the river bed. The tanks are filled with stones and immersed into the water in such a way that their upper ends protrude above the water level. To provide adequate room for a workman to operate, the clearance between adjacent tanks should be 55 to 65 cm. The stones are removed as soon as the tanks adfreeze to the ice. Freezing is accelerated when ice and snow are dumped into the water between the tanks.

Freezing of the water causes occurrence of pressure which may result in heaving of the ice. Therefore, it is advisable to chop several holes in the river ice at a distance of 80 to 100 m downstream of the point where freezing operations are carried out.

The amount of ground water can be reduced in some cases by means of frost belts. The frost belt is constructed normal to the ground-water flow and bars its access to the excavation. Sometimes it is feasible simply to surround the excavation with a frost belt, as shown in Fig. 148. For this purpose, a ditch is dug all around the excavation at a distance of several meters from it. The ditch is 1 m wide, while its depth ranges from 1.5 to 2.0 m, depending on the water table. In addition, a strip 1.5 to 2.5 m wide on each side of the ditch is cleared of surface cover.

The ground in the walls and bottom of the ditch freezes and merges with the impervious stratum of permafrost, rock, or clay, so that the ground water is prevented from reaching the excavation. If deep freezing is required, the bottom of the ditch can be deepened after it has frozen. This is done by successive removal of the bottom layers as soon as they freeze.

The rate of freezing is approximately 1.0 to 1.5 m every 15 to 25 days. It depends upon the texture of the ground, the moisture, and the temperature of the air. While freezing is in progress, it is necessary to remove the snow, water, and ice from the excavation and the stripped surfaces. The advantage of this method is that excavation in the pit proper is simpler and easier because it is done in thawed rather than frozen ground. Of course, this method is feasible only when the required conditions are favorable [27].*

B. Earthwork Operations in Frozen Ground

1. General Considerations and Specifications

Since construction on permafrost is particularly complex and it is impossible to plan earth structures in accordance with the totality of factors arising from local conditions and circumstances, construction practice should strictly adhere to the plans and should be in full accord with the local factors obtained from the available plan material and from special surveys when necessary. Accordingly, the construction engineer should be thoroughly familiar with the permafrost characteristics of the construction area involved, and should give careful consideration to both the design specifications and local conditions.

Excavation of frozen ground in cuts and pits in the permafrost region is quite complicated, very costly, and time consuming. The cost of such excavations often approaches the cost of similar operations in solid rock under ordinary conditions. This is primarily due to the fact that frozen ground is very strong. Some tests showed that the ultimate compressive strength of frozen ground was 100 to 120 kg per sq cm. Figure 149 vividly illustrates the strength of frozen ground. It shows an unsupported, thin arch of frozen ground, formed when gravel has been excavated. Another

* This reference contains valuable information about excavation operations in permafrost and water-bearing ground.

contributing factor to the high cost of excavation in frozen ground is the fact that many types of frozen ground tend to become slud-like when thawed and it is virtually impossible to retain them in place. It is practically impossible to walk or ride on some thawed ground. Both horses and men tend to sink into it. Figure 150 shows a muddy cut filled with slud-like ground.

The suitable method and season for excavations in permafrost depend firstly on the composition of the ground and secondly on the moisture content.* Either winter or summer excavation operations are permissible in the case of well-draining ground, such as pebbles, gravel, or coarse sand, although summer operations require measures for retarding the thawing process if the ground is excessively moist. Winter excavation of earth materials, that is, fine sand, silt, loam, and clay, is preferable. Summer excavation of such ground is permissible only when the ground is not supersaturated, that is, if the moisture content is less than 30 per cent.

Frozen ground may be excavated by means of the following methods:

(1) In the frozen state, by cutting and fragmenting tools.
(2) In the frozen state, by explosives.
(3) In the thawed state, by thawing with natural heat.
(4) In the thawed state, by thawing with fires.
(5) In the thawed state, by thawing with steam points.

These methods are discussed here in detail in order to give the reader the opportunity to become familiar with them, so that he can select the most suitable method for a given situation.

a. Method 1

Use of shovels is not feasible because frozen ground is so strong that shovels cannot penetrate it. Preheated shovels are used occasionally, but this procedure is disadvantageous because the shovels have to be reheated repeatedly, which consumes time and labor. In addition, two or three times the normal number of shovels are needed as well as extra workmen to maintain the fires, prepare firewood, and heat the shovels. Furthermore, heating rapidly damages the shovels. This method is applicable only in the case of excessively wet ground or soft, loamy ground if the volume of excavation is

* The bibliography at the end of this book lists references containing information about excavation operations in frozen ground.

relatively small. The advantage of this type of operation is that it is possible to excavate only as much ground as necessary, which is not feasible in the case of any other method of excavation in frozen ground. Moreover, this method does not disturb the permafrost regime.

The use of picks and crowbars for preliminary loosening of the ground is permissible but not very effective. It is difficult to break frozen ground, and much time and effort are needed to obtain results. On the average, one workman can excavate approximately 0.75 to 1.20 cu m during a normal workday if the ground is excessively wet, or 1.2 to 2.0 cu m if the ground is medium wet. Pneumatic tools are most effective in loosening the ground. However, these tools require compressors. (The aspects of excavation in frozen ground are discussed by F. N. Ron [28].)

Summer operations by the method under consideration are rendered considerably more difficult because the permafrost surface and the clods thaw rapidly and turn into mud so that the work proceeds in mud. The sides begin to slide into the cut or pit. Water from the thawing ground, sometimes supplemented by rain, accumulates in low lying spots (Fig. 151) so that thawing of the permafrost proceeds even more intensely. It is not always possible to divert the water, and often it is impossible to remove it by pumping. Similarly, it is impossible to use sheet piling. As a result, it is necessary to contend with a mire in which it may be impossible to walk or ride. Sliding of walls and slopes necessitates removal of much more ground than would otherwise be necessary. To avoid this, summer operations in excessively wet ground require measures to reduce the intensity of thawing of slopes in cuts. This is achieved by covering the slopes with peat or moss. In addition, it is essential to install drainage ditches in order to collect the water in one place from which it could be removed by one means or another. Figure 152 shows summer excavation operations with crowbars, picks, and shovels in a muddy cut along the Amur Railroad. The slopes of the cut slipped extensively in spite of shoring with posts and heavy planks. The track for small cars, laid over beams and logs, settled and deformed.

b. Method 2

Blasting operations in frozen ground are less effective than in normal thawed ground because frozen ground is highly plastic. However, this

method is most advantageous with respect to rapid loosening of large quantities of ground, which constitutes a requirement when the excavation operations are mechanized. Blasting does not affect the natural permafrost regime. Therefore, this method is preferred when it is desired to conserve the remaining permafrost after the required volume has been removed. According to available information, ammonal is more effective than dynamite because it loosens the ground more thoroughly. Ordinary blasting powder is unsuitable for use in frozen ground because small charges accomplish little and large charges blast the ground into very large chunks which are dispersed over a large area. A large number of simultaneous blasts is advantageous because it has the maximum pulverizing effect.

Blast holes are drilled in the usual manner. Manual drilling in frozen ground presents certain difficulty because frozen ground is extremely firm. herefore, it is advisable to use pneumatic tools. These tools accelerate the operations. Since frozen ground is quite hard, it is advisable to use drills of high-grade steel, such as Swedish steel or "Pobedit,"* or other particularly hard alloys. The blast holes are generally arranged as in the case of rock blasting.

If the work is conducted in the winter, it is advisable to insulate the air lines leading from the compressor to the drills. To prevent freezing of the water condensing in the air lines during interruptions in drilling operations, it is necessary to drain this water and to blow out the lines at regular intervals.

The blasted material is best removed by a power shovel or dragline excavator. The material can be hauled away by any means desired if the work is proceeding during the winter.

As in the previous case, considerable difficulties arise during summer operations in supersaturated ground, particularly if it consists of silt because the exposed permafrost surface thaws and the sides slide. Thawing of the slopes can be retarded by using a temporary protective cover of insulating material on the slopes. This cover may consist of moss, peat, straw, or brushwood, and should be 20 to 30 cm thick. The

* A Soviet alloy. — M.P.

water accumulating in a cut or excavation should be immediately removed by diversion or pumping.

c. Method 3

The use of natural heat for thawing frozen ground is feasible only during a few summer months, approximately from May or June to September or October, depending upon the region. This method is applicable only if the area involved is extensive and the required depth of thawing is not great.

This method operates as follows. The entire area is divided into two or three sections. The thawed top layer is successively removed from each section and the permafrost surface is exposed. The air warmth and solar heat act on the exposed permafrost surface and thaw it. As soon as a layer of 20 or 30 cm thaws out at the first section, this layer is removed and the underlying layer is given a chance to thaw. The work proceeds consecutively in each section, so that removal of the ground proceeds in one section while the second section is thawing out.

The main disadvantage of this method is its slowness. Thawing proceeds slowly, particularly if the ground is supersaturated. The intensity of thawing depends largely upon the temperature of the air, the composition of the ground, the moisture content of the ground, exposure of the area to the sun, etc. Thawing usually proceeds at the rate of approximately 15 cm per week inthe case of ordinary supersaturated permafrost consisting of silt. The rate of thawing is more rapid in the case of sandy ground.

The thawed ground usually become liquid, so that work must proceed in mud. Special plank roadways have to be installed to facilitate passage of dump carts. Often it is impossible to remove the ground with shovels, and it is necessary to use buckets. The walls of excavations and the slopes of cuts slide when thawed. Since shoring is difficult, this sliding involves excessive removal of ground even if special measures are taken to retard thawing of the walls and slopes. However, this method is quite suitable for sandy or gravellyground, particularly if this ground is not excessively wet, that is, if the moisture content does not exceed 30 to 40 per cent, as well as in the case when the work schedule is relatively

unrestricted with respect to time and the area involved is large.

It is rational to use the method of thawing with natural heat in conjunction with hydraulic dredging. The latter method is applicable only when conditions are suitable, primarily when it is feasible to remove the mixture of water and ground rapidly and readily by gravity flow to a place where accumulation of this water would have no harmful effect on the local permafrost conditions. Under favorable conditions, the combined method of hydraulic dredging and natural thawing is economical with respect to both time and labor. Since this work must be conducted during the summer, the required equipment does not differ from that used in similar operations under ordinary conditions.

d. Method 4

This method involved thawing of the permafrost by means of the heat of bonfires arranged on the permafrost surface. The procedure is as follows. The thawed layer overlying the permafrost is removed, and fires are built directly on the exposed permafrost surface. The heat from these fires thaws the permafrost. Usually the fires are made in the evening and maintained throughout the night. By morning the ground thaws to a depth of 20 to 50 cm. The thawed layer is removed during the day, and a new fire is built in the evening. Figure 153 shows the arrangement of an initial bonfire along the Amur Railroad for the excavation of a cut in May. The fire has just been started. In the background is a fire which has already burned out, so that removal and hauling of the ground is in progress.

According to Passek [26], bonfires for excavations are of t types. The first type is used in all cases except when the ground consists of clay. First, two logs are placed directly on the ground as shown in Fig. 154. These are the base logs. The space between these logs is filled with kindling (chips or deadwood), and firewood is placed on and normal to the base logs. The quantity of firewood depends upon the required depth of thawing and varies from two layers to a layer 1 m thick. Larch or birch is preferred. This type of bonfire is not effective

in the case of frozen clay because it causes thawing only to a depth of 20 cm. Therefore, another type of bonfire is used. It consists of a bonfire built of wood piled 1 m high, as shown in Fig. 155. When the bonfire is well under way, it is covered with a layer of manure about 10 cm thick and topped with a layer of snow. The fire is permitted to smolder for about 36 hr, while snow is added in places where it has thawed.

If the ground is excessively water bearing, it is necessary to prevent the ground water from extinguishing the fire in the pit. For this purpose, a part of the pit is excavated deeper, while the bonfire is arranged on the elevated part. The water accumulated in the depression is immediately removed by pumping or bailing. If the freezing method is used to deepen the excavation, the water should not be allowed to remain in the pit.

After the fire has burned out, the thawed ground is removed by methods depending on the moisture content of the ground. If the bonfire is arranged in a pit excavated by means of the freezing method, the thawed ground must be removed immediately after it has thawed. The thawed ground should not remain in place overnight because it may heat the entire frozen mass of ground and thereby cause flooding of the excavation by external water. For the same reason, it is inadvisable to deposit the warm, excavated ground in the immediate vicinity of the pit. This material should be deposited a reasonable distance from the pit.

The walls of pits in which bonfires are made should be protected by wooden shields suspended from stakes driven into the ground, as shown in Fig. 156. If unprotected, the walls might thaw excessively and slide. The shields are made of planks, and the bottom of the shield should be 80 to 90 cm above the bonfire.

The advantage of the method of thawing ground by fire consists of the fact that this method can be applied during the winter, so that this method can be used in conjunction with the freezing method. The following quantities of firewood (depending upon quality) or coal are required for thawing ground containing 60 to 70 per cent water:

(1) 0.7 to 0.9 cu m of wood or 130 to 150 kg of coal per cu m of

loamy ground.

(2) 0.4 to 0.6 cu m of wood or 80 to 100 kg of coal per cu m of sandy ground.

e. Method 5

The design and operation of steam points have been discussed previously. Thawing of the permafrost by means of steam points is most advantageous and, accordingly, is a highly recommended method. This method makes work simpler and easier and accelerates the thawing process. Operational costs are not high. This method has a certain disadvantage due to the fact that a considerable quantity of water is introduced into the ground. However, this is not always a matter of importance. Moreover, the water can be removed (Chapter IV-A). Excavation of ground thawed by steam points is conducted in the same way as in the case of ordinary thawed ground. Maximal utilization of mechanical equipment is recommended.

2. Special Instructions Regarding Construction of Fills

The best time for construction of earth fills in permafrost regions is during July and August, that is, when the active layer has thawed to a considerable depth. If it is necessary to conserve the permafrost beneath the fill, work should begin as early in the season as possible. Fills consisting of well-draining material and located on relatively dry ground comprising sand and gravel may be constructed during any season of the year. If constructed in the winter, the frozen ground may be used provided it is broken up into small pieces measuring 25 to 30 cm.

If need arises, fills may be made of frozen, wet earth materials or even silt, provided a special base of well-draining material is prepared and the following special measures are taken. Frozen earth material should be broken into fragments measuring 5 cm in diameter. The fragmented frozen ground should be deposited in layers not thicker than 40 cm each, compacting each layer by rolling or some other means. Every second layer of frozen ground should be topped with a 40-cm layer of well-draining material such as coarse or medium sand, gravel, or fine stone. The surface

of each double layer of frozen ground should have a slight outward slope (Fig. 157).

Engineer Kovalev used this method in erecting fills of frozen ground during the winter. The fill was 4 m high and designed to accommodate three tracks. The frozen ground consisted of supersaturated loam containing coarse gravel. A safety factor of 35 per cent was allowed for settling of the fill. The frozen material consisted of excavated chunks and was deposited in layers of approximately 40 cm each. The spaces between the chunks were filled with finer material. A layer of ballast, 20 to 30 cm thick, was placed on top of each layer of frozen ground until the fill was completed. The material was transported in dump carts drawn by horses. Kovalev maintains that the frozen ground should be fragmented if time does not permit letting the fill stand and thaw out. His experience indicates that fragmentation is not essential if the time element is not a factor. During the summer the fill settled about 42 per cent and the settling was not uniform. However, the fill dried during that time and was in good shape.

Kovalev also built a fill of frozen ground during the summer. The fill was 3.6 m high and consisted of loam containing coarse gravel. It was 1.2 km long. The frozen ground was deposited in thin layers, 10 to 30 cm thick, which thawed rapidly in the sun. Since the profile of the fill had steep slopes, the thawed material tended to slide but soon dried and formed a stable mass.

In preparing a base for low fills (up to 3 m high), it is necessary to remove the grass or moss cover from the ground surface within the base area. The peat occurring beneath the layer of moss should be removed only to a depth of 1 m. In the case of a fill exceeding 3 m in height, it is not necessary to remove the grass or moss cover and the underlying peat. The compaction of a thawed peat layer may be taken as 50 to 70 per cent of its original thickness.

Mechanical compaction of fills with steam or Diesel rollers is generally advisable and is essential in the case of silt loam. The fill material should be deposited in strictly level layers of about 30 cm each. It is advisable to divide the area involved in three sections so that while

the ground is deposited and leveled on one section, the second section is drying and the relatively dry ground on the third section is rolled.

3. Special Instructions Regarding Construction of Cuts

Summer excavation operations in permafrost comprising pebbles, gravel, and sand is not particularly difficult if adequate drainage is provided. It is difficult to excavate this ground, however, when it is frozen. Therfore, it is preferable to carry out these operations during the summer, utilizing the warmth of the air to thaw the ground or, if time does not permit, utilizing explosives.

During excavation of cuts it is essential to facilitate uninterrupted drainage of the water by installing ditches having a grade not smaller than 0.003. The dimensions and location of the ditches depend on local conditions. The work should proceed consecutively at a number of points and along the entire width of the cut, including the slopes. Upon completion of the slopes, they should be covered with a temporary layer of peat 0.25 to 0.30 m thick or a layer of moss 0.20 to 0.30 m thick in order to avoid inflow of excess water from the slopes and to retard thawing of the slopes.

It is advisable to use explosives for winter excavation of well-draining ground. Use of a power shovel on chain treads and dump trucks or tractor trailers is recommended, respectively, for loading and hauling away the blasted ground.

Either summer or winter excavation operations are feasible in the case of permafrozen loam and sand, loam which are not excessively wet and turn into a plastic mass of dough-like consistency. During summer excavation of such ground, it is essential to avoid direct effects of external load on the ground surface (movement of workmen, horses, equipment) because the ground tends to become excessively sticky and tacky. It is advisable to use a dragline or, if necessary, manual operation, utilizing dump cars. The ties beneath the dump-car tracks should rest not directly on the ground but on longitudinal logs (three or four) in order to distribute the load on a large area. Power shovels may be used during winter

operations. Loosening of the ground can be accomplished by means of wedges, explosives (preferably), or bonfires.

If bonfires are used, it is necessary to prepare a narrow trench with vertical walls. The initial width and depth of the trench should be 1 m and 0.8 m, respectively. Subsequently, in order to facilitate the operation, the bonfires should be arranged on the bottom of the trench near the wall and the excavation should be gradually widened as it is deepened. In the case of excessively wet fine-textured material which tends to become relatively liquid when thawed, it is advisable to conduct excavation operations during the winter if it is not feasible to avoid cuts in such a ground even when the route is relocated. The drains in the slopes and the deep drainage ditches, designed in accordance with Fig. 115, should also be installed in the winter.

Ground of this type may be excavated with picks, crowbars, or explosives. Blasting with ammonal is the best method. Thawing of supersaturated ground by bonfires is not recommended because the excessive moisture interferes with proper functioning of the fires while the thawed ground, transformed into a liquid, necessitates removal with scoops and makes traffic on its surface difficult.

In addition to the methods already discussed, the hydraulic method of excavating cuts in frozen ground is advantageous when the conditions facilitating the use of the method are available.

In all cases of summer excavation of cuts in frozen ground, the slopes should be covered with a temporary layer of moss or peat, 20 to 30 cm thick, in order to retard thawing. Final reinforcement of the slopes of cuts excavated during either summer or winter should be carried out during the following autumn because the slopes would dry and acquire relative stability during the intervening season [29, page 196].*

CHAPTER VI

CONSIDERATIONS REGARDING OPERATION OF STRUCTURES ON PERMAFROST

A. Special Comments

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* This reference contains additional information about excavation of cuts and erection of fills on permafrost.

The preceding analyses and discussions make it clear that the safety of many types of structures in the permafrost region depends to extent upon a number of special measures which are taken in order to pr serve and create the particular situations and conditions under which t occurrence of phenomena peculiar to this zone would have the least harm effect on the structures. However, the equilibrium established in this way is relatively unstable and can readily be disturbed, with the resul that most of these measures would become useless, the conditions would change, and the structure would be endangered.

Evidently, both proper selection of a construction site and s able methods of design and construction often are insufficient because subsequently, in the course of time during which the structure is expec to function, a change may occur in the factors and conditions which det the design and construction of the given structure. Under certain cond tions, this change may affect the safety of the structure. Accordingly the completed and submitted structure should be under constant and care observation, and its operation must be in strict accord with the local conditions upon which the design and construction were based. Thus, th safety aspect is involved not only in the surveys, design, and construc but is essential during the entire existence of the given structure. T is, this aspect should be the concern of the persons operating the stru ture as well as of the construction engineers.

It is possible to site many instances in which structures tha were relatively correctly designed and constructed began to deform bec of improper operation, without due consideration of the local pecularit and the design factors and conditions. Accordingly, as soon as a struc is being utilized, it is necessary to keep it under observation while taking all the steps necessary to assure and facilitate maintenance of factors and conditions upon which the design was based. For this purp when a structure is submitted to the operating organization, it is ess to submit also the detailed design and construction data which characte the given locality, the construction area, and the structure itself, a which include all the special measures taken to assure the safety of th

structure under the given conditions. The operating organization should utilize these data to establish a corresponding regime of maintenance and operation.

The preceding statements refer to all types of structures, including buildings, bridges, fills, dams, cuts, etc. In this connection, the OST Manual No. 90032-39 offers the following instructions, based on the particular importance of proper operation and maintenance of structures in the permafrost region.

OST NO. 90032-39

VII. MAINTENANCE OF STRUCTURES

39. When structures are erected on permafrost, it is essential to take instrument readings of the deformation both during construction and during the first years of operation. These observations are carried out in accordance with a special schedule attached to the plans and modified in the course of construction. These observations are compulsory in the case of second-class structures and in the case of major and typical structures of the third class. In newly developed permafrost areas, this procedure is advisable even in the case of temporary structures.

40. When a structure is completed and submitted, the operating organization should request and the construction firm should submit the data collected during the surveys and investigations of the construction site. The deed should contain a note regarding the transfer of these data. The following supplementary material pertaining to the construction site should be included: geology, hydrogeology, permafrost regime, physical and mechanical properties of the ground, chemical properties of the ground water, load tests, etc.

B. Instructions and Considerations Regarding Proper Operation of Structures

The operation of any structure erected on permafrost should be in complete accord with the construction method involved (Chapter IV,A). All the measures incorporated in the plan for the purpose of improving the effectiveness of the structure should be maintained and developed. Thus, in the case of a structure erected in accordance with method A, that is, on the principle of conservation of the permafrost at the base, the upper permafrost limit should be checked two or three times each year during operation of the structure--in the spring, in the summer, and at the beginning of the first frosts. The results of these observations should be recorded so that the behavior of the permafrost could be traced.

Ventilated air spaces beneath heated buildings should be opened for the winter and closed during the summer. At any rate, they should be utilized for regulating the state of the permafrost. In some cases it may be unnecessary to open these spaces during the winter, at least during extreme cold, as in the case of the previously mentioned house at Skovorodino.

The major contributing factor to instability of the permafrost layer is water with its high heat content. Therefore, special care is required to prevent seepage of atmospheric or industrial water into the ground near and beneath the building. For this purpose, all water diversion and drainage facilities, as well as the installations designed to prevent seepage of water into the ground, should be kept in good order and regularly repaired. To facilitate transfer of cold to the ground, it is advisable in some cases to remove the snow near a building during the winter because snow cover heats the ground and prevents the transfer of cold to the ground.

Backfill and cushions beneath and near a building, as well as the vegetation, should be maintained in the same condition as that specified in the construction plan. In the case of alterations of functional changes in a building, consideration must be given to its design and to the measures taken during construction, so as to avoid disturbing the conditions determining the safety of the structure.

Direct local heating of walls or foundations by installations such as furnaces, forges, or steam pipes is not allowed except at points specified in the original plans. Similarly, in the case of a completed building erected on the principle of permafrost conservation, neither utilization of the ventilated air space for any purpose whatever nor digging of cellars, wells, or pits is allowed because these procedures would cause thawing of the permafrost. Large quantities of hot slag, cinders, manure, or other waste should not be deposited near buildings.

If a given construction method has been used to erect an existing structure, the safety of this structure would be endangered if different construction methods are used in erecting nearby structures.

Proper regime of operation and observation of deformations is required also in the case of buildings erected by the active construction method. In this case, as in the preceding instance, all special installations specified in the plan and carried out during construction should be properly maintained in order to retard thawing of the permafrost and thus reduce the magnitude and nonuniformity of settling. If settling occurs, it is necessary to apply the specified measures to eliminate its consequences, utilizing at the right time the devices for regulating the elevation of various structural elements. These devices include blocks, wedges, etc.

Installations designed to prevent heaving, as well as other protective installations, require definite maintenance. Such maintenance primarily involves keeping all installations in working order and use of timely repairs. The extent of swelling depends primarily on the rate of inflow of water. Therefore, it is essential to prevent every type of backfill from becoming saturated with water, so that the backfill would not swell. The water which can be diverted should be prevented from seeping into the ground near the structure. Whenever feasible, it is essential to prevent the ground water from reaching the ground in the vicinity of the building. It is perhaps advisable in some cases to recondition nonswelling backfills by partial replacing of the fill material with new material at regular intervals or by periodic soaking of the backfills with oil waste or naphtha.

Certain maintenance is required also in the case of road structures such as filter dams, culverts, trestles, and bridges erected on timber or masonry supports. In the case of filter dams, it is essential to make sure that any water which may have accumulated upstream during the autumn would not be blocked off and remain there to freeze during the first frosts. If such freezing occurs, the dam would not function the following spring. In addition, the installations designed to prevent silting of the dams should be properly maintained. In the case of any road structure, including fills, constructed on the principle of conserving the permafrost at the base of the structure, it is essential to prevent any protracted accumulation of water near the structure in order to conserve the permafrost. Water has a high heat content. Therefore, when water accumulates near a structure, it

may readily cause deep thawing of the permafrost and deformation of the structure. Accordingly, it is essential to facilitate free flow of the water. Accidental causes of water accumulation should be eliminated as rapidly as possible. Therefore, drainage installations should be kept in perfect order. Ditches, culverts, and similar drainage facilities should be regularly cleaned and their design grades should be maintained, so that no stagnation water would occur.

Erection of a structure may result in swampiness of individual areas in some cases. This should be avoided, using appropriate measures, if the excessive moisture in the ground would tend to cause percolation of water into the swelling ground near the structure. In other cases, swampiness may facilitate rising of the upper permafrost limit.

When fills are erected on frozen ground, it is essential to avoid flooding of the barrow pits and accumulation of water in them. Water in the pits may cause deep thawing of the permafrost. As noted in Chapter IV,E frost belts designed to check the progress of icings and to reduce the moisture in the active layer require special maintenance. "Seasonal" frost belts, consisting of a strip of ground free of snow and a snow dike on the downhill side, require regular removal of the snow after every snowfall. According to Chekotillo, permanent frost belts in the form of ditches cease functioning after three to four years if improperly maintained. Maintenance of such a ditch consists of covering the bottom of the ditch with a layer of moss or peat in order to prevent deep summer thawing of the ground beneath the bottom of the ditch. This insulating layer should be removed for the winter in order to facilitate rapid and intensive freezing of the ground beneath the bottom of the ditch. In addition, permanent frost belts should be freed of snow after every snowfall in order to intensify freezing of the ground. Snow removal should end in February because it is no longer necessary. Experience has demonstrated that ground frost belts function faultlessly if they are properly maintained. The moss or peat layer which is removed for the winter should be saved and utilized during the following summer. It is necessary to remove the snow also from the wings of the frost belts after every snowfall.

To allow free passage of the icing water beneath a bridge, the

installations for insulating stream beds (Chapter IV-E) should be erected promptly at the beginning of each winter and removed in the spring. The materials involved should be stored for use during the next year.

CHAPTER VII

ASPECTS OF ANALYTICAL AND EXPERIMENTAL STUDIES

A. General Remarks

The student of construction on permafrost will find in the relatively limited literature on this subject many unclear passages, much uncertainty, and frequently acknowledgement that various aspects are inadequately clarified. The literature in this field contains numerous contradictory opinions of persons who are interested in the practical aspects of construction on permafrost.

Numerous aspects of this interesting and complex field actually remain inadequately investigated and verified, although the majority of these aspects could have been resolved long since by means of uniform experiments and observations arranged simultaneously in various permafrost regions. The reason why many practical problems of construction on permafrost have not been solved as yet by means of observations of adequate duration and by means of an adequate number of experiments can be attributed primarily to the fact that no program or exact formulation of procedures has been available and because the dispersed efforts in this field have not been unified and properly directed.

There exist in our country the most diverse types of scientific research institutions, the objectives of which are strictly practical—to assist actual enterprises and to further the development, improvement, and simplification of a given industry. After extensive experimentation and research, these institutions produce new designs, apparatus, processes, and methods which can be utilized in practice and are adopted in industry. Moreover, these developments are well established and verified. All these technological and industrial achievements are the result of a fairly long process of research in a given problem and do not constitute conclusions drawn from occasional or, in general, accidental observations and experiments

or research. On the contrary, they are always based on numerous observations and experiments which supplement, complement, confirm, or refute one another.

Such is not yet the case as far as construction on permafrost is concerned. Despite the fact that certain problems were posed quite a long time ago, they have not been subjected to either systematic verification or systematic investigation. Many important aspects of construction on permafrost, some of which could be readily solved, remain controversial or were rather improperly solved because of lack of a theoretical basis and an adequate number of indisputable practical experiments and observations. Unfortunately, a number of problems have been solved, on the basis of isolated experiments or accidental facts. Thus, for example, even such a problem as the use of heated buildings with ventilated air spaces, the solution of which is presently regarded by many engineers as quite satisfactory, has been inadequately investigated and its solution is actually based only on the data obtained from observations of a single wooden house at Skovorodino.

The current solution of another major problem that of control of icings by means of frost belts, is based on Petrov's careful observations and tests conducted in the course of only one winter, and perhaps on some indirect data. The magnitude of the heaving force on columns and foundations still is an open question because Bikov's solution (the force is 120 kg per linear cm of the column perimeter) is applicable only to a specific area at Skovorodino, since this solution is based only on a few experiments conducted during one winter and utilizing the same local ground of definite texture and a given moisture content.

It is possible to list numerous other aspects in the field of construction on permafrost which are solved either on the basis of accidental data or on assumptions. The incorrectness of this procedure needs no proof. Unfortunately, absolutely no alternative is available at present. Experience of previous years and the work of various former ~~and present~~ institutions for experimental research demonstrate that none of these institutions has found a complete solution for certain problems of construction on permafrost. The results achieved in individual cases do not measure

up to the tasks involved, do not correspond to the efforts made, and do not entirely satisfy actual requirements. The reason probably is that the budget allotted for experimental research was divided among various offices each of which pursued its own narrow objectives. Consequently, planning has been inadequate and the results frequently remained unpublished.

Numerous practical questions remain unanswered depite the enormous advance in the science of permafrost, in comparison with its status prior to the revolution, due to the accomplishments of the Academy of Sciences. Many organizations and offices established permafrost stations in the various permafrost regions for the purpose of finding solutions for particular problems. These stations were poorly equipped, staffed, and financed, and they had improper guidance. Most of them lasted only a short time, while others functioned intermittently with frequent changes in personnel, so that many investigations were started but not completed. Frequently, expeditions were outfitted for the purpose of studying aspects of construction on permafrost. However, the expeditions usually had a definite time schedule which did not permit them to do all the necessary work because it would have required stationary installations and a series of repeated experiments. With rare exceptions, the results obtained by such expeditions have not been of major value as far as practical construction is concerned.

The practice or organized permafrost stations is a good one, but such stations require constant attention and care. Their work should follow a single plan and should be under unified and planned supervision.

It seems obvious that clarification of all these aspects requires systematic and planned work of an independent organization which would concern itself primarily with construction problems. The colossal size of the permafrost area, which constitutes almost 47 per cent of the entire territory of the USSR, and extensive construction in this area require establishment of a special organization for experimental research devoted solely to problems of construction on permafrost. The establishment of such an organization, perhaps in the form of a scientific research institute dealing with aspects of construction on permafrost, would make it feasible to bring together the dispersed specialists in this field, to unify their

activities, and to initiate planned and supervised practical studies of the problems of construction on permafrost. This organization would be equipped to assemble and analyze all available materials and data. It would be in a position to solve rapidly the urgent construction problems arising in actual practice, which remain unsolved because of the absence of such a specialized organization.

B. Cardinal Construction Problems Requiring Solution

The essential problem which has not been satisfactorily solved as yet is the magnitude of the heaving force on columns, piles, and foundations It would be feasible to obtain an approximate solution, adequate for practical calculations, in the course of two or three winters if a series of experiments, indentical in method and instrumentation, were carried out with the objective of determining the magnitude of the heaving force in various types of ground at various permafrost regions. Simultaneous thorough tests to determine the aspects of freezing of the active layer and the heaving of supports would yield more exact information regarding the adfreezing strength between the active layer and the supports and concerning the thickness of the active layer. This problem has not been solved primarily because correct, sufficiently extensive, and planned experiments have not been carried out thus far.

Experimentation along these lines should proceed in accordance with the instructions presented in Chapter III,B-3b, utilizing at first the instruments recommended by the Skovorodino permafrost station [30]. Separate tests should be conducted on timber and concrete supports located in various types of active layer overlying permafrost occurring at various depths.

The problem of protecting structures against heaving is of equal importance. The effectiveness of nonswelling backfills around foundation supports—consisting of gravel, slag, or similar material, as well as the advisability of soaking these materials with oil waste or naphtha—has not been determined as yet. Many engineers contend that these backfills are valueless and do not reduce heaving of the supports because they rapidly become silted and the ground water rapidly dilutes the oil. It is true

that none of the opponents of utilizing backfills offers proof as to where, when, and under what conditions a definite structure has been subject to heaving despite such a backfill. Nevertheless, this opinion persists, and certain organizations ceased using such backfills.

Tsytovich's laboratory tests (Table IV) confirm to some extent the fact that backfills undergo intensive swelling. On the other hand, it is known that backfills function excellently in the case of numerous structures. The construction engineers who built the old railroads (the Amur, Trans-Baikal, and other railroads) regarded backfilling as a highly satisfactory means for prevention of heaving. Numerous articles in current literature indicate that backfills often help to safeguard structures against heaving. Thus, all the buildings in the region of the Amderma River, rest on wooden posts passing through an intensely swelling active layer about 1.50 m thick. Some of these supports have been backfilled with gravel and the corresponding buildings are in good condition, while the buildings which rest on posts that have not been backfilled underwent extensive heaving.

It is quite obvious that this problem could be readily settled if studies were organized of existing structures at various places and if a number of experiments were conducted under various conditions and with various types of backfill. Observations conducted over a period of several years would definitely eliminate all doubtful aspects. The Skovorodino Station, which was established several years ago, could have long since conclusively solved this problem.

It has repeatedly been recommended to counteract heaving of building and bridge supports by anchoring the supports in the permafrost or in the ground beneath the active layer. However, no practical data are available as yet regarding the cases in which anchoring is required and the manner in which it should be applied. These aspects require an adequate number of experiments which would yield basic results that could be applied at least in the most typical cases.

It is equally essential to determine the effectiveness of inclined piles and posts which do not heave ordinarily, as indicated by available observations.

The use of ventilated air spaces beneath heated buildings erected in accordance with the principle of permafrost conservation has been recommended previously. This recommendation was based on the theoretical considerations evolved by Tsytovich and Sungir and was substantiated by a limited number of factual data. Unfortunately, Bikov's observations disprove the orderly theory of the temperature regime in a heated building. In order to obtain a final practical and theoretical solution of this problem, at least as far as the most common cases are concerned, it is necessary to conduct a series of special observations of buildings with ventilated air spaces. The method for construction of buildings adapted to settling due to thawing of the permafrost, recommended by the OST Manual No. 90032-39, is not sufficiently definite and can hardly be utilized in practice because it is impossible to estimate correctly the magnitude of settling of the ground. N. A. Tsytovich recently made a successful attempt in his doctoral dissertation to develop a theoretical solution to the problem of settling of both uniform and stratified types of ground due to thawing of the permafrost. His method is confirmed by results of laboratory tests. However, it is necessary to verify this method under actual conditions and to adapt it to practical facts. Proper experimentation at permafrost stations would furnish the ultimate solution to this problem, which is one of the most important in the field of stable construction.

The use, design, and operating regime of the steam point has not been adequately clarified, although this device is most suitable for use in frozen ground. Extensive utilization of the steam point on construction necessitates further practical study of the aspects pertaining to application of this method.

It is important to solve the problem regarding the time required for restoration of the permafrost thawed by the steam point during the process of lowering piles. This requires basic tests and observations. Only scattered data are available at present, which complicates the task of the construction engineers.

Mention was made previously regarding the advisability of utilizing refrigeration procedures in some cases in order to conserve the

permafrost. Utilization of the reserves of natural cold for this purpose is highly promising. However, it is imperative to develop this method under field conditions and by means of suitable experimentation. It is opportune to begin to deal with this important problem.

The foregoing does not cover all the aspects which are of primary interest to construction engineers. The aspects presented here comprise only those which require relatively immediate solution and which can be rapidly resolved. Of course, the activities of a special institution for scientific and experimental research in construction on permafrost cannot be limited solely to investigations of individual construction problems, but should cover the entire problem. This activity required special and more profound analysis.

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APPENDIX

INSTRUCTIONS FOR INVESTIGATION OF STRUCTURAL DEFORMATION

The following are excerpts from the 1939 Draft of the Provisional Engineering Standards for surveying, design, and construction of railways on permafrost, NIIPS NKPS.

A. General Instructions

1. The object of the investigations is to determine the factors and causes of deformation and correspondingly to develop measures for protecting the structures against the deforming effects of permafrost phenomena.

2. Investigations of structural deformation should comprise the following: (a) engineering aspects, including the details of design, construction, and deformation; (b) permafrost aspects; (c) geological and hydrogeological aspects.

3. Investigations of structural deformation should be conducted by a committee consisting of competent persons and including at least the following three specialists: (1) a construction engineer who is familiar with the aspects of permafrost, (2) a permafrost scientist who has a knowledge of structural engineering, and (3) a geologist specialising in engineering geology and familiar with permafrost and hydrogeology. If the scope of the investigations and the size of the structures are relatively small, the investigations may be entrusted to a few persons or even to a single individual, provided this individual is sufficiently competent in the special fields involved.

4. The investigations of structural deformation may be divided into the following two categories: (a) general investigation of the condition of individual structures or groups of structures, and (b) investigation of deformation and its causes. The second category can be subdivided as follows: (1) general investigation of deformation, (2) detailed study, and (3) long-term investigation. In addition, the activities involved comprise the following: (a) inspection of buildings, (b) inspection of bridges and culverts, and (c) inspection of roadbeds.

5. The particular type of investigation involved is determined in accordance with specifications presented in the following paragraphs.

B. General Inspection of Structures

1. The objective of the general inspection is to determine the general character, causes, and factors involved in the deformation of structures in the given region.

2. The general investigation and its scope are as follows:

(a) A general description of the area is prepared. This description should contain detailed climatological and meteorological data (precipitation, snow cover, temperature regime, etc.) which can be obtained at local institutions, from various climatological handbooks, and by interviewing local residents. The description should contain the general characteristics of the region with respect to geology, hydrogeology and permafrost conditions, as well as information regarding the local topography, vegetation, and surface cover. This description must be based on direct observation and familiarization with available materials, and should utilize the procedures specified in (d) below;

(b) A general description of the design and appearance of the deformed structures is prepared on the basis of personal observations;

(c) A description is prepared of the nature of the deformation, indicating the types of deformation prevailing in the given region, such as settling, heaving, etc. This description must be based on personal observation of the structures. It is advantageous to include photographs of all the deformed structures and brief comments regarding the particular design and deformation. In describing the deformation, it is essential to indicate the location of the structure with respect to geographical latitudes and longitudes.

(d) If feasible, it is advisable to drill or dig test holes near the structures having the most typical deformation. This should be done near three to five structures at the points of most extensive deformation, using one hole or bore per structure.

3. The records of this investigation should comprise the following: (a) general description of the locality: climate, meteorology, geology, hydrogeology, permafrost, and vegetation; (b) general description of the structures and the nature of the deformations involved; (c) photographs of the locality and the deformed structures; (d) description of the measures undertaken to counteract the permafrost effects on the structures, the effectiveness of these measures, and the opinions of the investigators regarding the probable causes of deformation; (e) logs or index cards containing the results of drilling or excavation operations, if such operations were performed; (f) the opinions, conclusions, and recommendations of the investigators regarding the nature and extent of further studies.

C. General Investigations of the Deformation of a Specific Structure

The procedures and scope of the general investigation of deformation are as follows: (1) A general description is prepared of the area where the specific structure is located, in accordance with the instructions presented in "B" of the foregoing; (2) A detailed description of the design aspects of the given structure is prepared on the basis of personal observation; (3) A detailed description of the extent and nature of the deformation is prepared in accordance with inspection results and is accompanied by photographs of the structure; (4) The opinions and conclusions of the investigators regarding the deformations and their causes are prepared, indicating the extent of any necessary further investigation.

D. Detailed and Long-Term Investigations of Deformation of an Individual Structure

1. Both the detailed and long-term investigations of structural deformation are carried out in accordance with the following instructions, the latter investigation having a relatively wider scope, as indicated later in the appropriate paragraphs.

2. The program and scope of the investigations are determined in accordance with the results of the preliminary investigations specified in "B" and "C" of the foregoing.

E. Investigation of Deformation of Buildings

1. Investigation of the engineering aspects should comprise the following procedures and scope:

(a) Accurate sketches of the building, to a 1:50 scale, are prepared in accordance with tape measurements. They comprise the following: (1) a plan of the building foundations and of each story, (2) general longitudinal and transverse sections indicating the type and depth of the foundations, and (3) cross sections at points where deformation occurred.

(b) These drawings should be accompanied by a description of the design and layout of the building, and the construction materials used for various parts of the building. The locations of the deformations are indicated in the drawings. In addition to the careful description of the deformations and their extent, it is essential to describe with precision and in detail the design and materials of the foundations and their bases, the construction methods, and the procedures and measures applied during construction for the purpose of avoiding the effect of permafrost on the structure. The information can be obtained from working drawings, inspection of the structure, and interrogation of local residents.

(c) The location of the building with respect to geographical longitude and latitude should be specified.

(d) The character of the area is described, indicating the topography, vegetation, and surface cover; and a topographic map is prepared, drawn to a 1:500 scale, showing the area around the structure within a radius of at least 100 m.

(e) The measures for diverting the surface water from the vicinity of the building are described, and a schematic plan of the system of diversion and drainage ditches is appended.

(f) A detailed description of the deformations is prepared, and the deformations are indicated on the plans and section of the building.

(g) The dimensions of the various deformations, that is, the length, width and relative location of cracks, the extent of settling or heaving, etc., are prepared and special sketches are drawn if necessary.

The cracks are measured with a yardstick and sliding calipers. The boundaries of the cracks are marked on the structure with oil paint. Settling and heaving are determined with the aid of a level. The readings are compared with those given in the original plans of the working drawings. If those original data are not available, the readings are compared with those relating to the undeformed adjacent portions of the building, such as the opposite wall or side.

(h) The investigators should supplement the records with their opinions and conclusions regarding the aspects listed previously as well as any other aspects which they consider important and valuable.

(i) In the case of long-term observations of a deforming structure, the progress and relation of the deformation to external phenomena should be described systematically and precisely.

2. The nature and scope of the geological, hydrogeological and permafrost investigations are as follows:

(a) A general description is prepared of the geological, hydrogeological and permafrost characteristics of the region, including a general description of the locality, the nature and peculiarities of the permafrost, the composition of the active and permafrost layers, the moisture content of these layers, and the temperature and physical and mechanical properties of the ground.

(b) The stratification and nature of the ground near and beneath the building are determined.

(c) The regime, level, and chemical composition (aggressiveness) of the ground water are established.

(d) The upper permafrost limit and the thickness of the active layer at the time of the investigation are determined in accordance with the procedures presented in (j) below. These measurements are made at 10 or 12 points in places where deformation occurred and generally near or beneath the building, and section profiles of the ground are prepared.

(e) The position of the upper permafrost limit during construction of the building is determined from the blueprints or inquiries.

(f) The maximum, minimum, and mean annual temperatures of the area are determined from appropriate sources, and complete observations of the climatic regime are conducted in the case of long-term investigations.

(g) The depth of the original and current snow covers in the given area are determined.

(h) The amount of annual precipitation is established.

(i) The mechanical strength of the permafrost and the adfreezing strength between the active layer and the foundation are determined.

(j) Obtaining the data involves the following steps. Three or more test pits, extending to the foundation base, are excavated at points where deformation occurred and several boreholes, extending at least [illegible] m below the foundation base, are drilled in order to obtain samples of the ground and to determine the depth of the permafrost. The number of boreholes should be sufficient to allow preparation of longitudinal and transverse profiles of the ground beneath the building.

(k) General conclusions are compiled regarding the probable causes of the deformation, as well as the nature and extent of the deformation and the preventive procedures.

(l) Analysis is made of aspects not listed in the foregoing, but regarded by the investigators as important.

(m) The following steps are imperative in the case of long-term observation of the deformation of a given structure: (1) to establish beacons and benchmarks and record the corresponding observations, (2) to conduct systematic observations of the temperature regime of the ground and the elements of the building. These observations should be carefully related to the temperature regime of the outside air and to the operational regime of the building, and (3) to determine the moisture content of the ground in the vicinity of the deformations at various depths ranging to 2 m below the foundation base, in the case of sudden occurrence of extensive deformation.

(n) If buildings exist in the given area, the investigators should inspect them, determine the design of the buildings and their

foundations, establish the nature of the deformation, if any, and the procedures which were used to prevent the effect of the permafrost.

F. Investigation of Bridge Deformation

1. The sequence and scope of the investigations pertaining to the engineering aspects are as follows:

(a) Drawings of the bridge or culvert should be prepared with adequate accuracy to a scale of 1:50 or 1:100, utilizing tape measurements of the structure or data obtained from appropriate offices. These drawings should include the plan of the supports, longitudinal and transverse sections of the supports and foundations, and the design and depth of the supports.

(b) The drawings should indicate the material of the various elements of the supports and the surface condition of the section of the foundation located within the active layer. The drawings should be accompanied by a detailed description of the design and arrangement of the bridge supports, with particular reference to the foundations and the procedures used during construction and subsequent operation of the bridge to prevent or reduce the effect of permafrost on the bridge. The points at which deformation occurred should be indicated on the drawings.

(c) In addition, the following information should be provided: (1) the facing material of the support and the color of the stone, (2) the fill material, (3) the design and cover of slopes, (4) the presence of shade and vegetation near the supports, (5) the design and effectiveness of the regulating and reinforcing installations. These data can be obtained from inspection.

(d) The location of the bridge with respect to geographical longitude and latitude should be indicated on the drawings.

(e) To describe the nature of the river and prepare a sketch of the effective channel, indicating the water levels at low stage, ice flow, and high stage (during floods or summer and fall inundations). It is essential to collect the results of hydrometric observations during the preceding five to ten year.

(f) To conduct a topographical survey of the vicinity of the bridge, covering an area in accordance with the size of the bridge, and describe the nature and contour of the locality.

(g) To prepare a detailed description of the deformations, indicating each deformation on the drawings.

(h) To make the following exact measurements of the deformations: (1) the length, width, and relative location of cracks, measured with the aid of a yardstick and sliding calipers; (2) the tilting or displacement of the structure or its elements; (3) the extent of settling or heaving. The last two measurements should be obtained by means of precise level readings. These readings should be compared with the benchmarks established by the local highway department and with the original blueprints. A transit should be used to survey curves, slopes, and displacement points, and the location of the supports should be carefully measured and checked. Sketches should be prepared if they can clarify the aspects of the deformations.

(i) The investigators should offer their views with regard to both the aspects enumerated here and any other aspects which they deem important and valuable.

(j) In the case of long-term observations of structural deformation, it is essential to establish benchmarks and beacons and to describe systematically and accurately the course of the deformation and its relation to external phenomena, using various strain gages as well.

2. The geological, hydrological and permafrost investigations comprise the following:

(a) Preparation of longitudinal and transverse geological profiles of the river bed on the basis of data provided by the department of roads or by means of field tests if necessary. The profiles should cover 100 m on each side of the bridge.

(b) Preparation of a general geological, hydrogeological and permafrost description of the locality, including a general description of the area, the nature and properties of the permafrost, the composition of

the permafrost and active layers, the moisture content of these layers, and the temperature and physical and mechanical characteristics of the ground.

(c) Determining the regime and level of the ground water by means of inspection, observation, and inquiries.

(d) Determining the occurrence of matrix or bedrock near the structure from data furnished by the organization in charge of the road or from field tests if necessary.

(e) Determining the position of the upper permafrost limit and the thickness of the active layer at the time of the investigation. These measurements should be taken at the following points: in places where deformation occurred--near the sides, front, and rear of the pier, and near the sides, front, and rear of the abutment (at least at four places, exclusive of those where deformation occurred). It is also essential to determine the thickness of the permafrost at the given area.

(f) Determining the position of the upper permafrost limit during construction of the bridge.

(g) Determining the maximum, minimum, and mean annual temperatures from appropriate sources.

(h) Determining the depth of snow cover from the data of a climatological reference book.

(i) Determining the amount and time of precipitation in the given area and conducting a complete series of climatic observations in the case of long-term investigations.

(j) Determining the mechanical strength of the permafrost in its frozen and thawed states.

(k) Determining the adfreezing strength between the permafrost and the foundations.

(l) To obtain the data enumerated in the preceding, it is necessary to carry out the following operations: (a) to establish test pits extending to the foundation base near places where deformation occurred;

(b) drill holes should be made in the test pits near the foundations, extending at least 5 m below the foundation base, depending on local conditions; and (c) the required number, dimension, depth, and distribution of the drill holes are determined by field conditions and should be such as to facilitate preparation of longitudinal and transverse geological and permafrost profiles.

(m) Determining the presence, location, formation period, and extent of ground icings in the vicinity of the bridge.

(n) Preparing conclusions regarding the probable causes of the deformations, their nature, sequence and course, and the necessary preventive measures.

(o) Presenting the views of the investigators regarding aspects which they consider important but which have not been included in the preceding.

(p) The following procedures are essential in the case of long-term observations of deforming structures: (1) to establish benchmarks and beacons and to keep a record of the observations, describing the circumstances, sequence and extent of the deformations, utilizing appropriate instruments; (2) to make systematic records of the temperature regime of the ground, the supports, and the outside air. The moisture content of the ground at points where deformation occurred should be determined at various depths ranging to 2 m below the foundation base if sudden occurrence of extensive deformation is evidenced.

(q) If other structures exist in the given area, it is necessary to describe them and their design, to determine whether they have deformed, and to ascertain the nature of the measures used to prevent the effect of permafrost on these structures.

G. Investigation of Deformation of Roadbeds and Drainage Installations

The investigation program and scope pertaining to the engineering aspects are as follows.

1. A longitudinal profile and a series of transverse profiles of the

roadbed area under consideration should be prepared from measurements and level readings. The transverse profiles are required for each benchmark and station indicated on the original longitudinal profile, and should comprise not only the roadbed itself (cut or fill) but also the berms, drainage ditches, bankets, shoulders, hillside ditches, and other drainage installations. Each profile should cover a distance of about 50 m from each edge of the roadbed. The longitudinal and transverse profiles should be drawn to normal scales. The profile of the normal (undeformed) roadbed, taken from the working drawings, is plotted along these profiles, using dashed lines. In addition, a topographical map, drawn to a scale of 1:500 and covering a minimum width of 100 m, is appended to these profiles.

2. The drawings should indicate the location, type, material, and condition of the shoring for fills, cuts, and diversion and drainage ditches.

3. Separate drawings should be made of all deformations, settling, cracks, slides, washouts, cavings, and similar displacements, relating them to the longitudinal and transverse profiles and indicating the date of their occurrence.

4. To prepare a list of the swellings in the roadbed, indicating their exact location and dimensions on the topographic map.

5. To describe the measures taken during construction and subsequent operation for the purpose of preventing deformation and stabilizing the roadbed and other earth structures in accordance with the design and specifications.

6. To furnish the data regarding the material, construction method, and time of erection of the fill, as well as similar information regarding the cut.

7. The investigators should state their opinions regarding the causes of deformation and the suitability of the corresponding preventive measures.

8. The geological, hydrological, and permafrost aspects of the investigations comprise the following procedures:

(a) To prepare a longitudinal, geological section of the roadbed and of its natural base beneath one of its edges, using test holes.

(b) To prepare a series of geological sections of the roadbed and underlying natural surface, using test holes made at the locations of the transverse profiles and other characteristic points.

(c) To prepare a series of longitudinal and transverse sections determining the presence and extent of ballast deposits, using test holes.

(d) To determine the level and regime of the ground water.

(e) To determine the geological and hydrogeological aspects of the area, which may have a negative effect on the stability of the roadbed.

(f) To prepare a general description of the geology, hydrology, and permafrost condition of the area, indicating the nature and properties of the permafrost, the composition of the active and permafrost layers, their moisture content, temperature, and physical and mechanical characteristics.

(g) To determine the position of the upper permafrost limit and the thickness of the active layer at the time of the investigation, within a section comprising the berms, ditches, and shoulders, and extending a distance of 50 m from each edge of the roadbed.

(h) To determine the position of the upper permafrost limit during construction.

(i) To determine the presence and extent of ice wedges, individual ice lenses, and masses of imbedded ice in the ground.

(j) To determine the maximum, minimum, and mean annual temperatures of the locality.

(k) To determine the type of natural surface cover on the area under investigation, as well as the time and extent of its disturbance.

(l) To ascertain when the woods, if any, were eliminated in the area involved.

(m) To determine the effect of the remaining woods (in individual places of the area involved) on the ground-water level, the ground moisture, and the position of the upper permafrost limit.

(n) To determine the thickness of the snow cover and the time of major snowfalls, as well as the amount and time of precipitation in the given area.

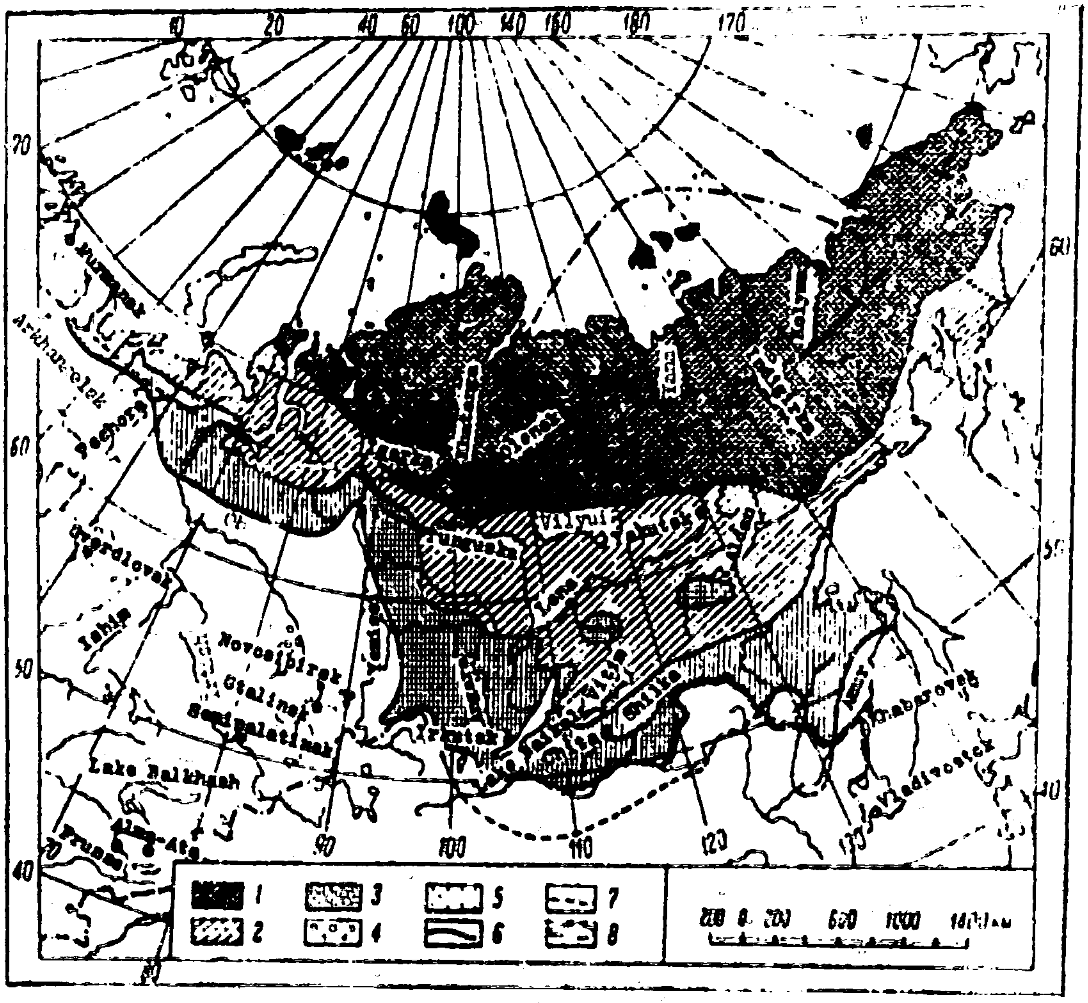
(o) A complete study of the climate is essential in the case of long-term observations.

(p) To determine the strength of the ground in compression, tension, and shear.

(q) A record is kept of the long-term observations, in which are described the time of occurrence and the nature, course, and extent of the deformation, as well as the circumstances accompanying the deformation.

(r) It is essential to make a thorough and detailed investigation of any other area which is not subject to deformation or undergoes negligible deformation, if such an area occurs in the vicinity of the deforming roadbed area, in order to determine the aspects (topography, geology, hydrology, permafrost, design, type of ground, etc.) which prevented deformation.

(s) The investigators should state their views regarding any important aspects which have not been included in the foregoing.



1,2,3. Ground temperature 10 to 15 m below the surface generally is lower than -5 C, varies from -1.5 to -5 C, and is higher than -1.5 C, respectively. 4. Isolated islands of permafrost. 5. Small permafrost lenses in peat mounds. 6,7. Southern boundary of permafrost within and outside (hypothetical) the USSR, respectively. 8. Permafrost area with thick inclusions of ground ice.

Fig. 1 – Schematic Map of Permafrost Distribution, Prepared by M. I. Sumgin in Accordance with Permafrost Temperature

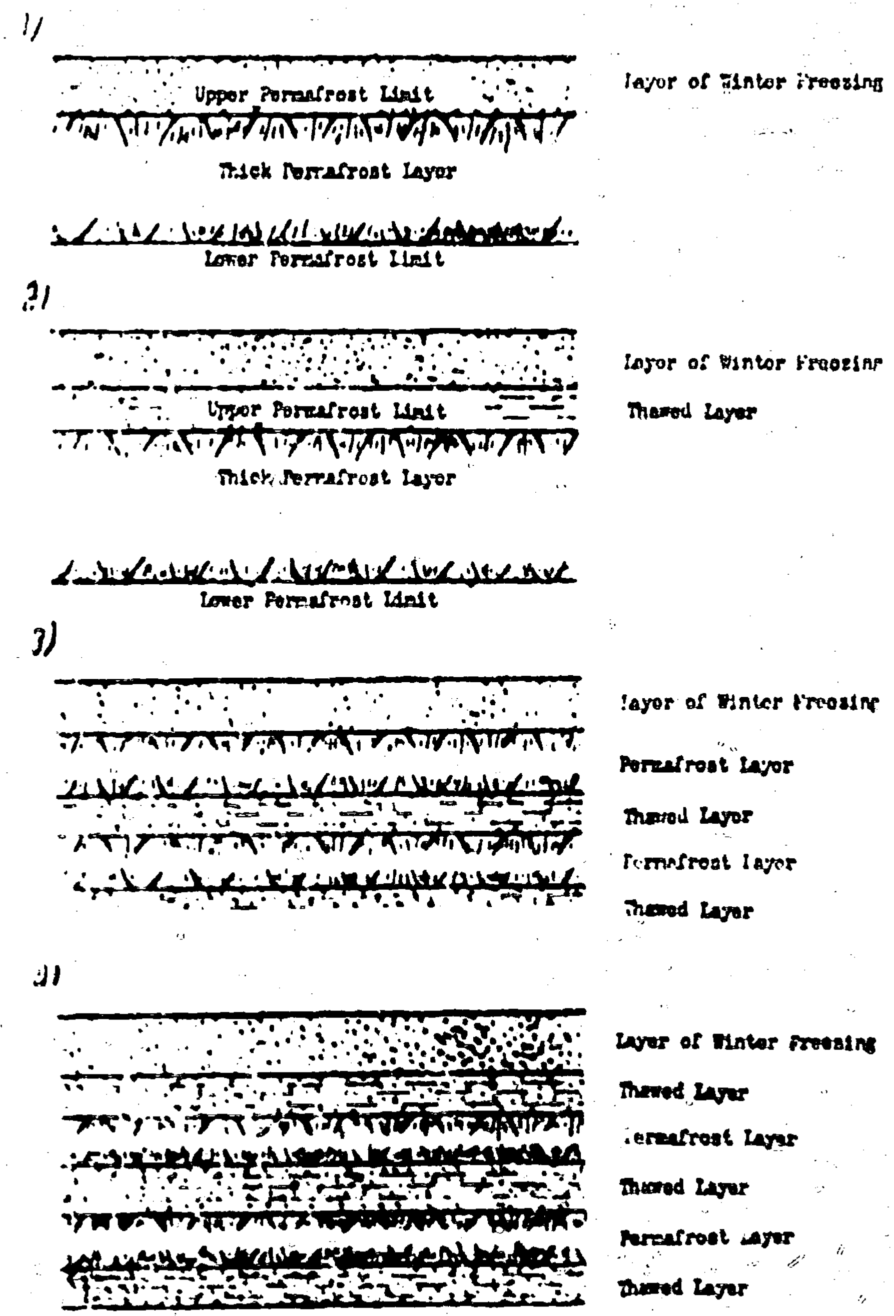


Fig. 2 - Schematic Geological Profiles of the Four Permafrost Types

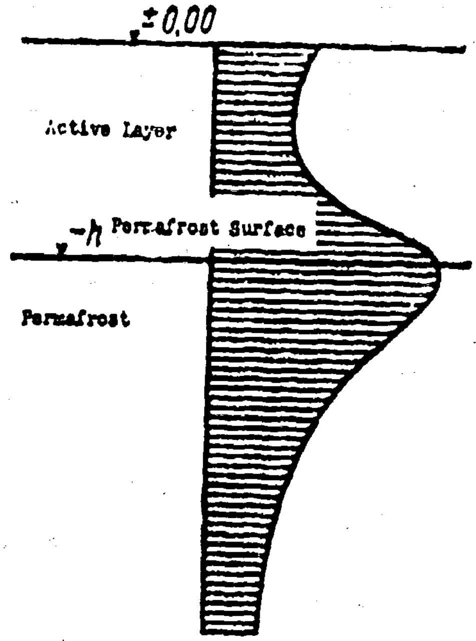


Fig. 3 - Typical Moisture Distribution in Active and Permafrost Layers

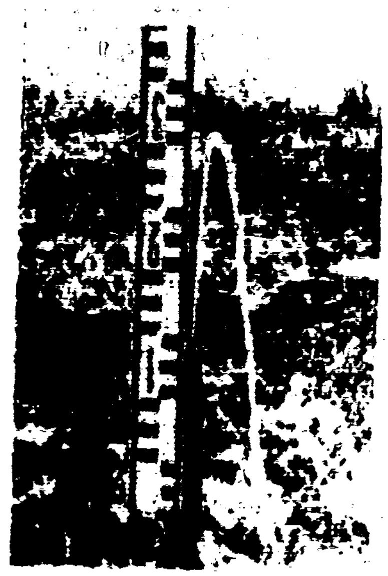


Fig. 4 - Ground-Water Fountain Exceeding 1 Meter in Height

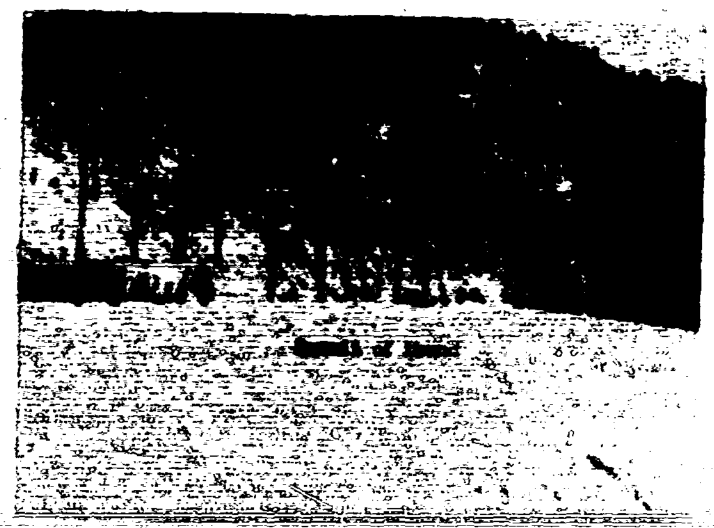


Fig. 5 - Summit of River Icing Mound near a Road

Fig. 6 - Ice Slab Weighing 38 Tons Ejected by Exploded Icing on the Onon River (From V. G. Petrov)

Fig. 7 - Ground Icing near a Railroad (Photographed May 28)

Fig. 8 - Ground Icing Mound near a Railroad Station

Fig. 9 - High Ground Icing Mound near a Railroad

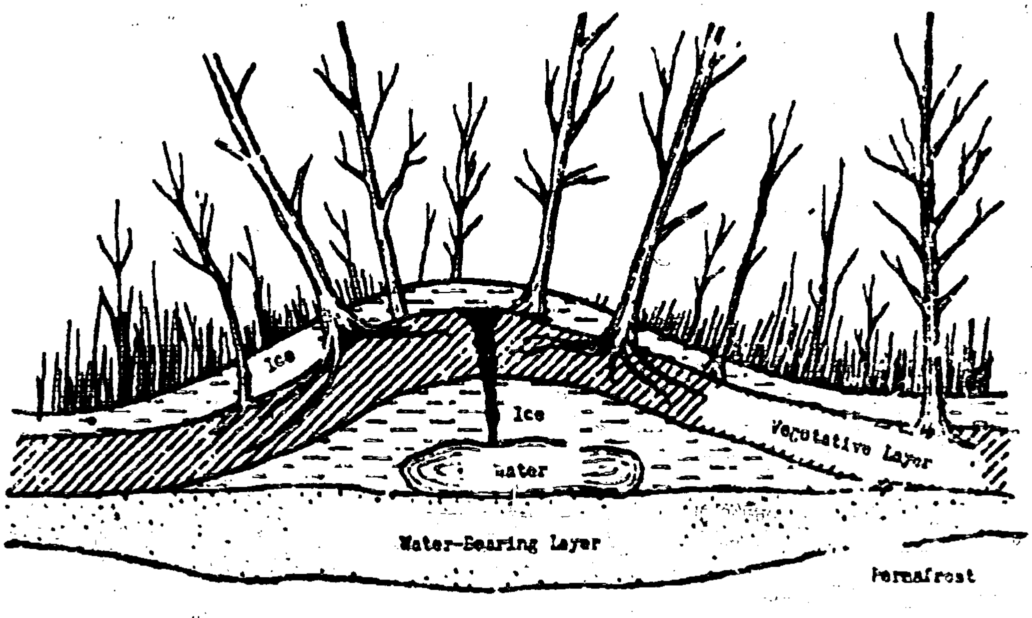


Fig. 10 - Diagram Illustrating Formation of Ground Icing

Fig. 11 - Deeply Ruptured Summit of Ground Icing Mound

Fig. 12 - Spring Icing Progressing Towards the River

Fig. 13 - Icefall in a Railroad Cut

Fig. 14 - Frost Mound in Yakutia (From P. I. Melnikov)

Fig. 15 - Ice Lens Beneath Thin Layer of Peat and Moss in a Bog

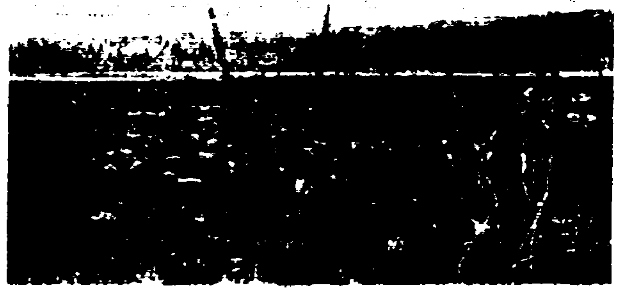


Fig. 16 - Hummocky Bog

Fig. 17 - Bouldery Alluvial Talus

Fig. 18 - Bare Bouldery Talus near the Niman River

Fig. 19 - Moss-Covered Bouldery Talus near the Niman River

Fig. 20 - A River Freezing Solid During Winter

Fig. 21 - The Niman River

Fig. 22 - Windfall in Siberian Forest where Permafrost Occurs at Shallow Depth

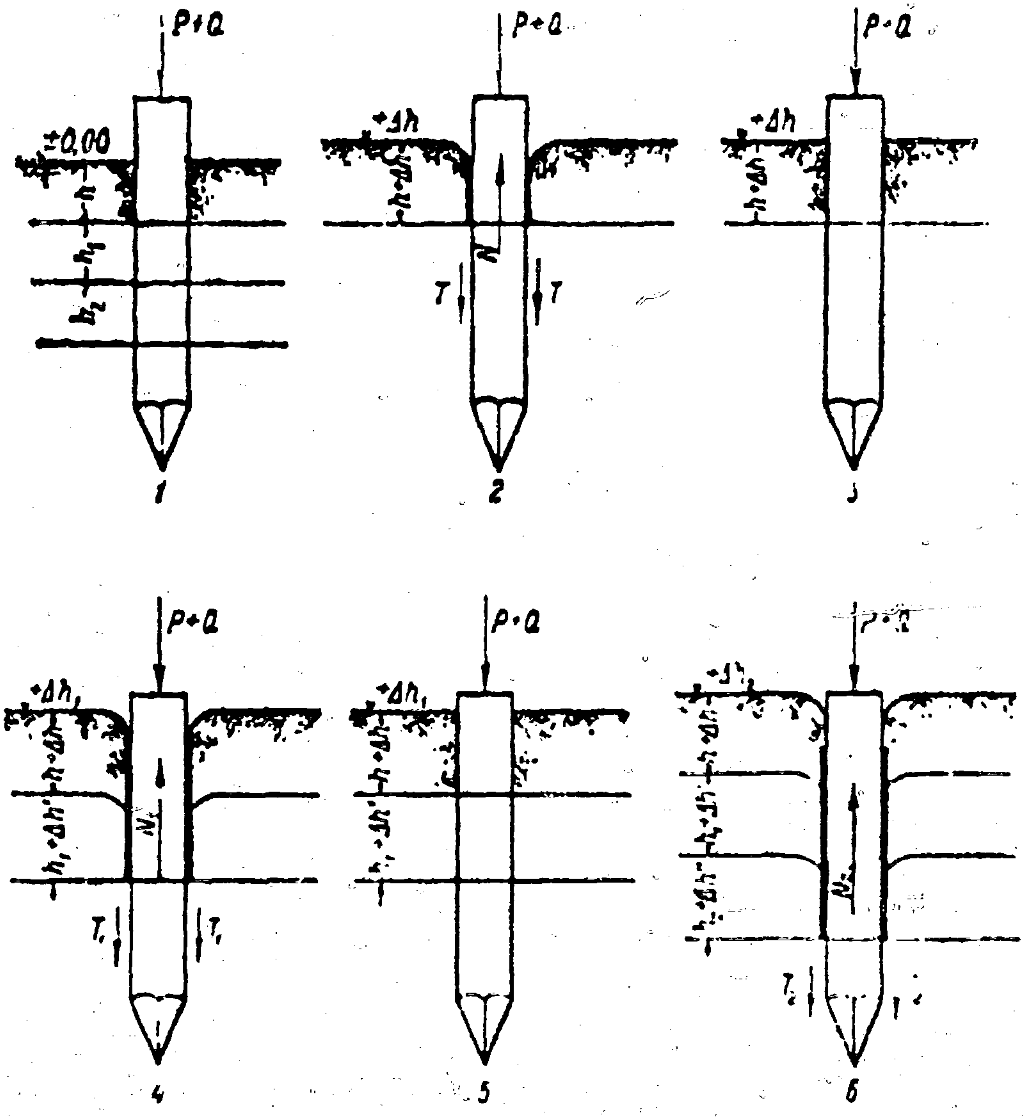
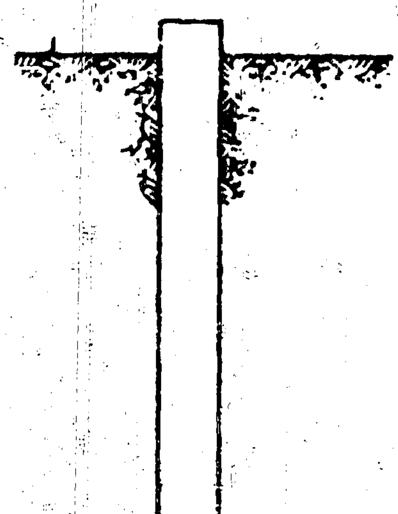
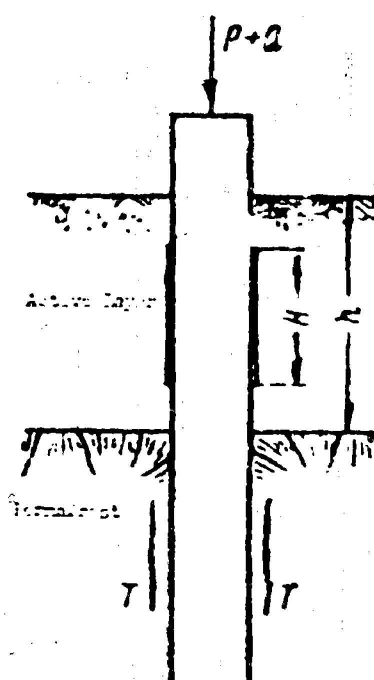


Fig. 23 - Diagrams Illustrating Effects of Frost Heaving

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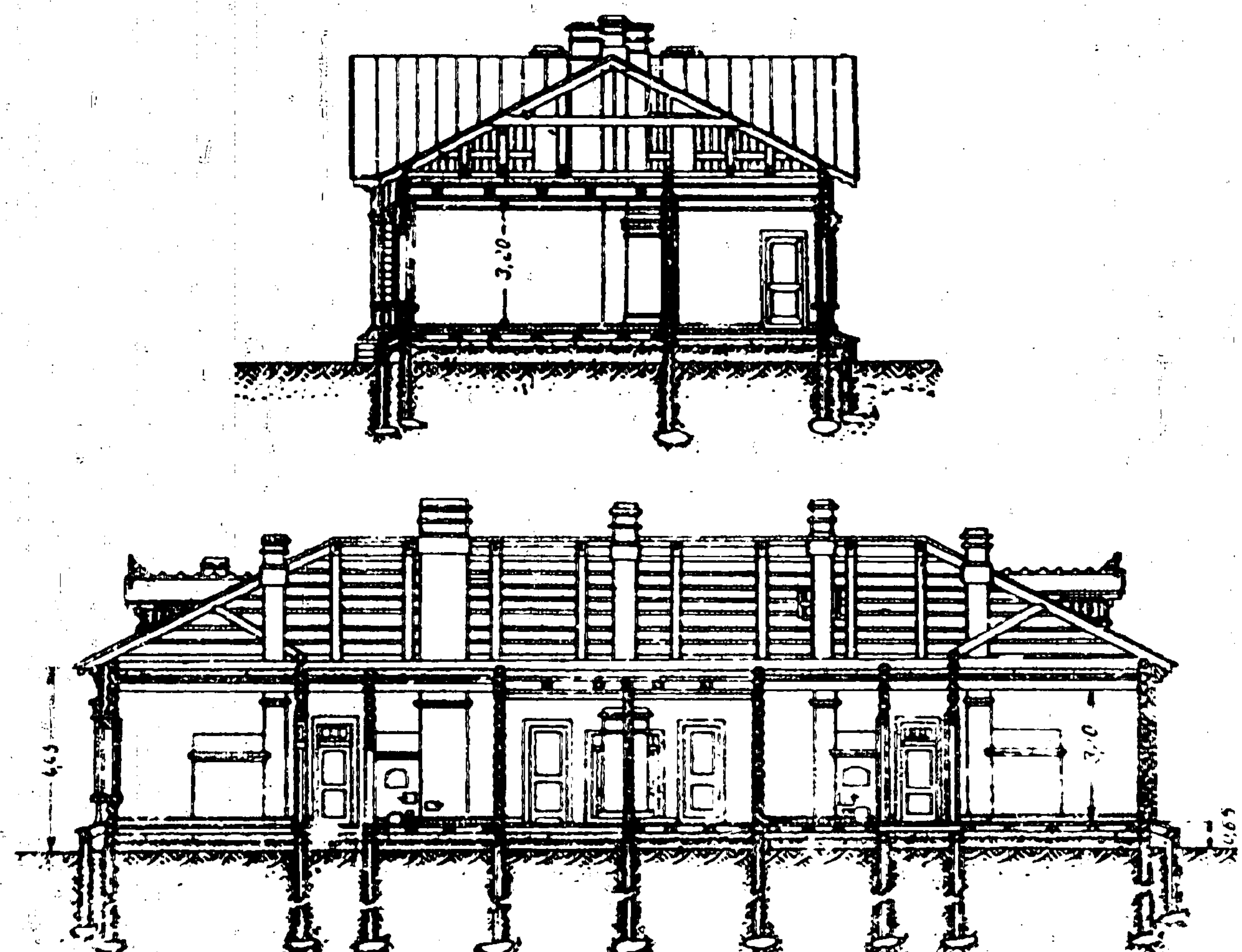


Fig. 27 - Typical Design of a Wooden Building on a Foundation Unsuitable for Swelling Ground

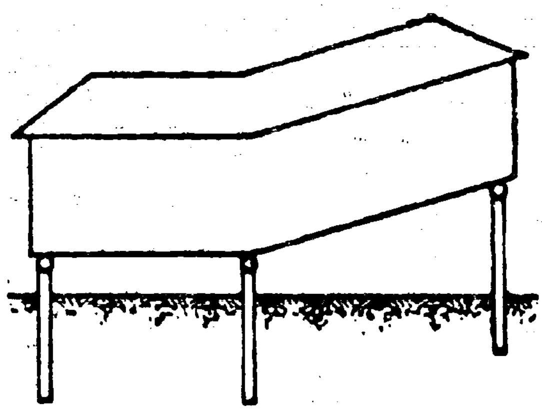


Fig. 28 - Deformation Pattern of a Wooden Building on Posts

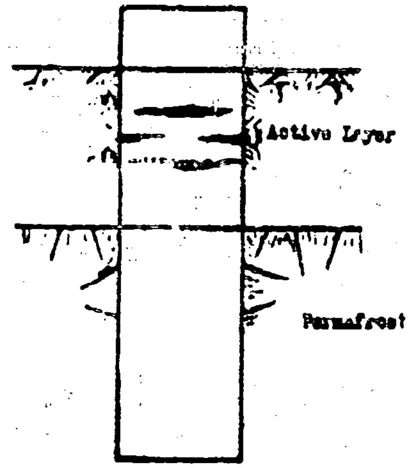


Fig. 29 - Masonry Column Failure Due to Heaving

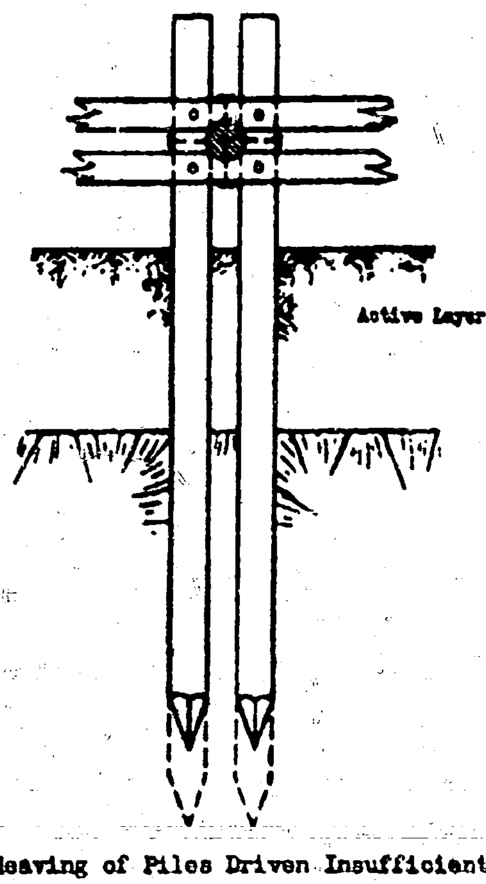


Fig. 30 - Heaving of Piles Driven Insufficiently Deep into Thawed Ground Beneath Active Layer

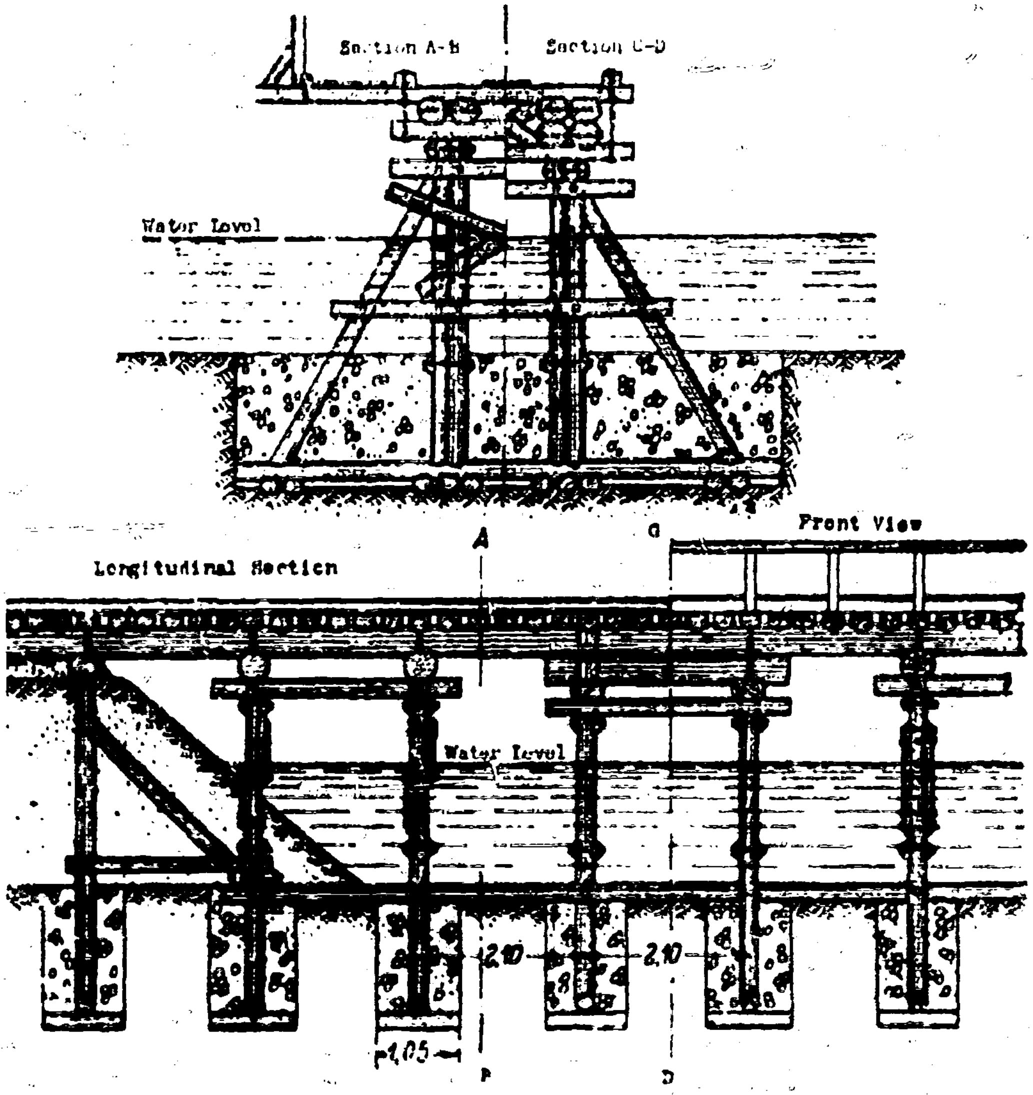


Fig. 31 - Unsatisfactory Design of Bridge Supports on Timber Grills in Swelling Ground

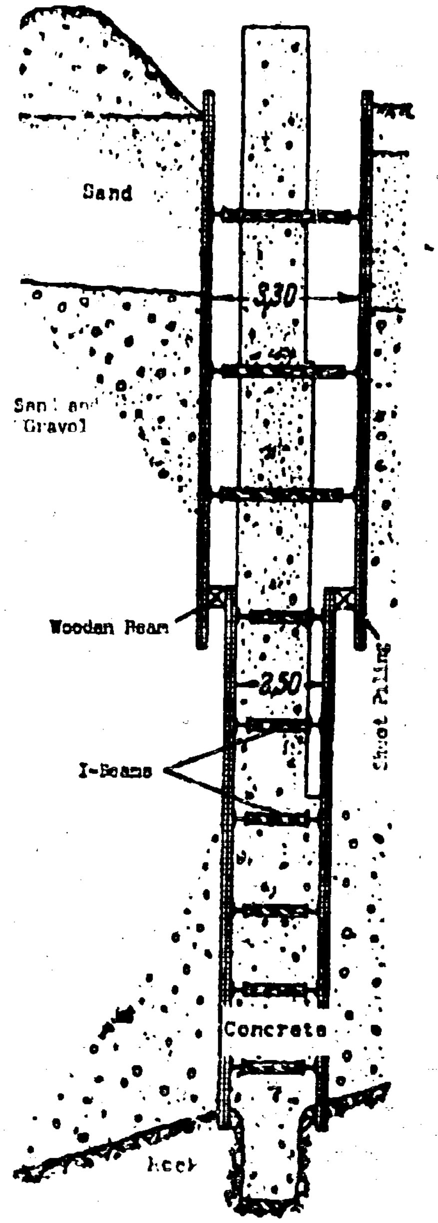


Fig. 100 - Foundation Column in Deep Pit with Sheet Piling

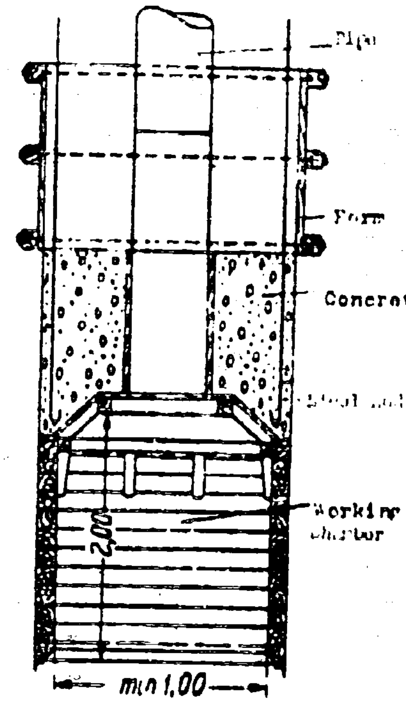


Fig. 101 - Typical American Caisson for Building Foundations

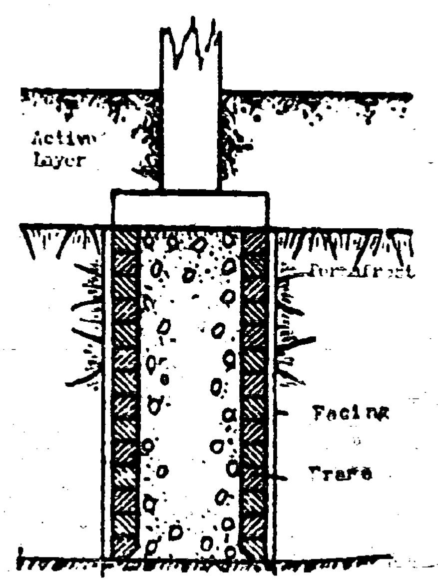


Fig. 102 - Timber Sunk Well

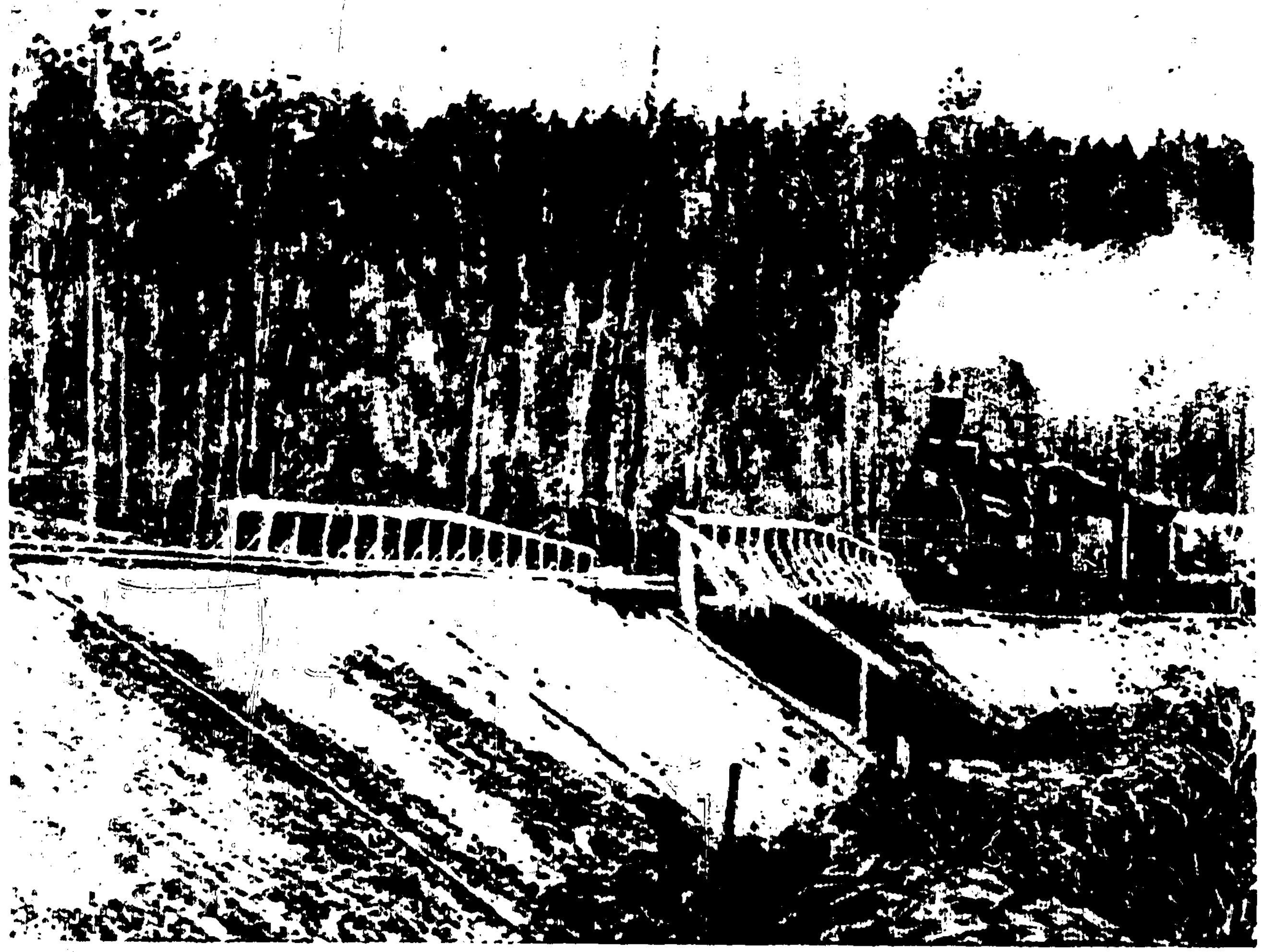


Fig. 34 - Hump Formed in Bridge Due to Heaving of Center Piles

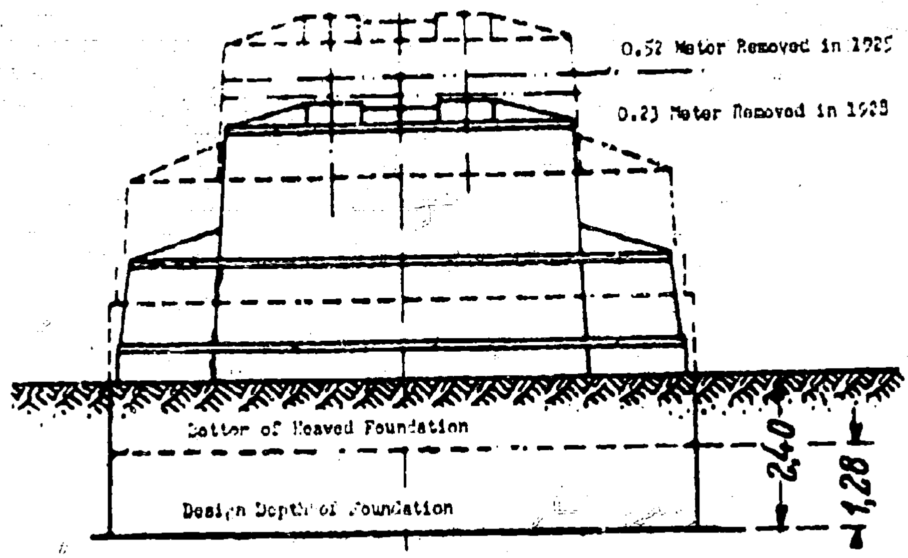


Fig. 35 - Heaving of a Pier

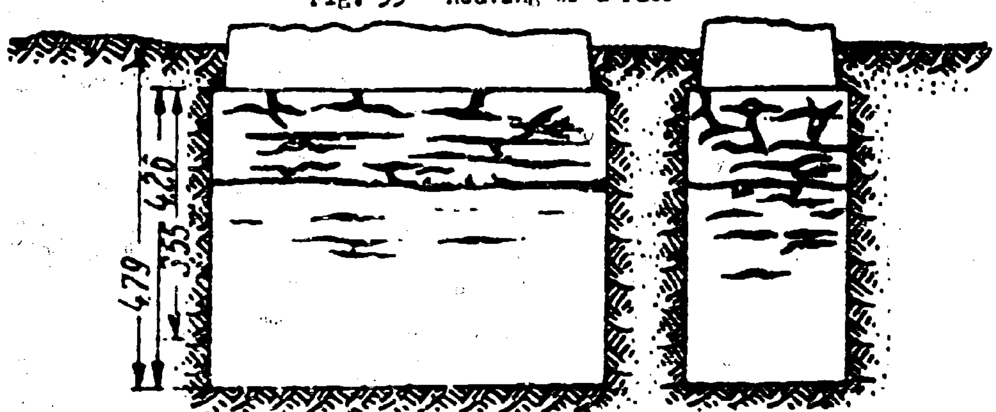


Fig. 36 - Pier Masonry Failure Due to Heaving

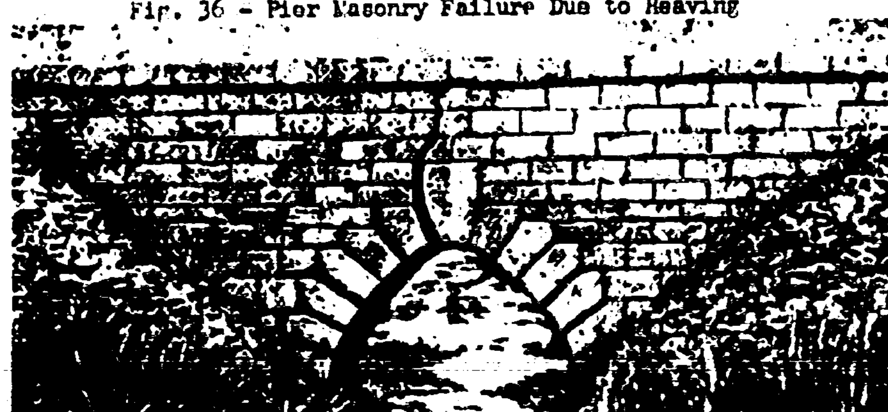


Fig. 37 - Culvert Deformed by Heaving

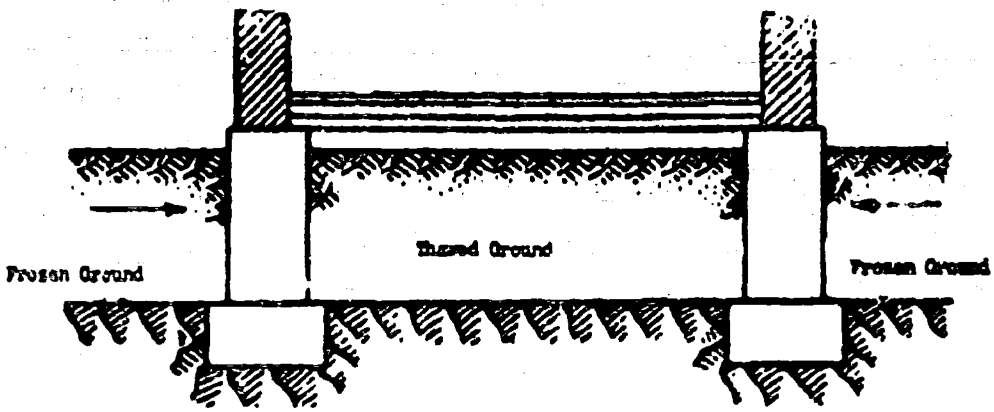


Fig. 38 - Foundation Displaced Horizontally by Swelling Ground

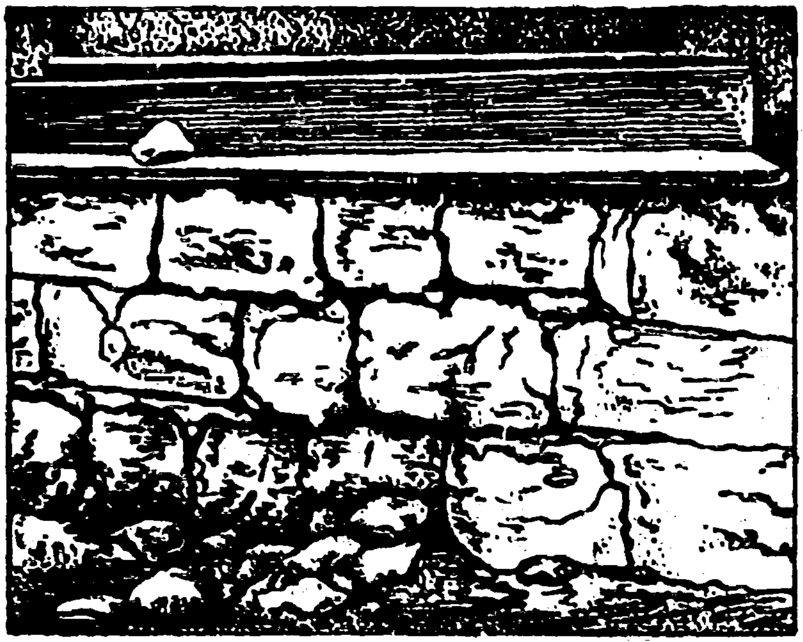


Fig. 39 - Foundation Deformed by Horizontal Pressure
(From M. I. Evdokimov-Rokotovsky)

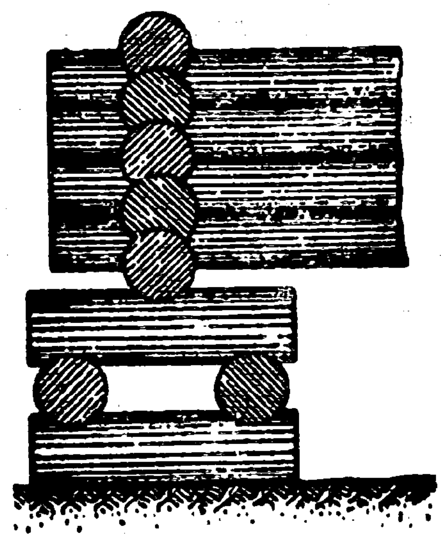


Fig. 40 - Building on Cribs

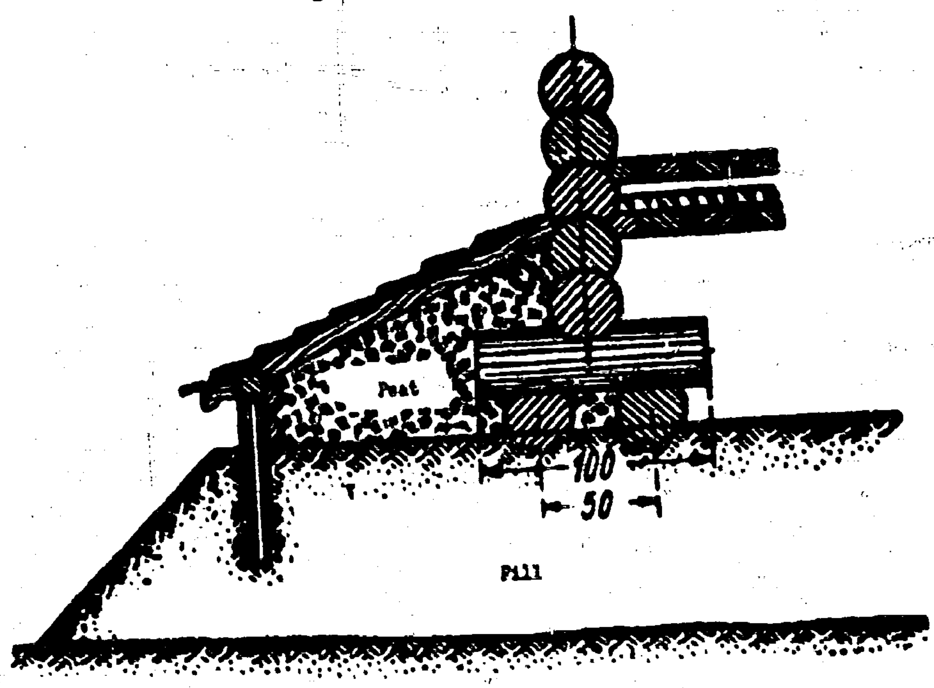


Fig. 41 - Crib Foundation on Fill

Fig. 42 - Slumping Caused by Melting of Imbedded Ice (From V. V. Elenevsky)

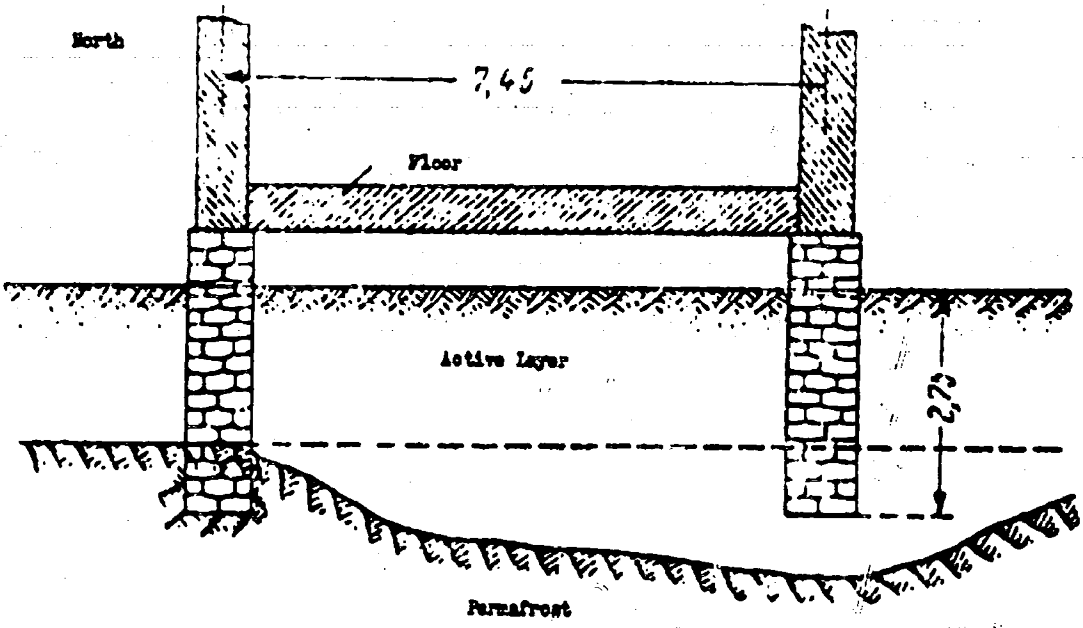


Fig. 43 - Lowering of Upper Permafrost Limit Beneath Experimental Masonry Building at Petrovsk-Zabaikalsk

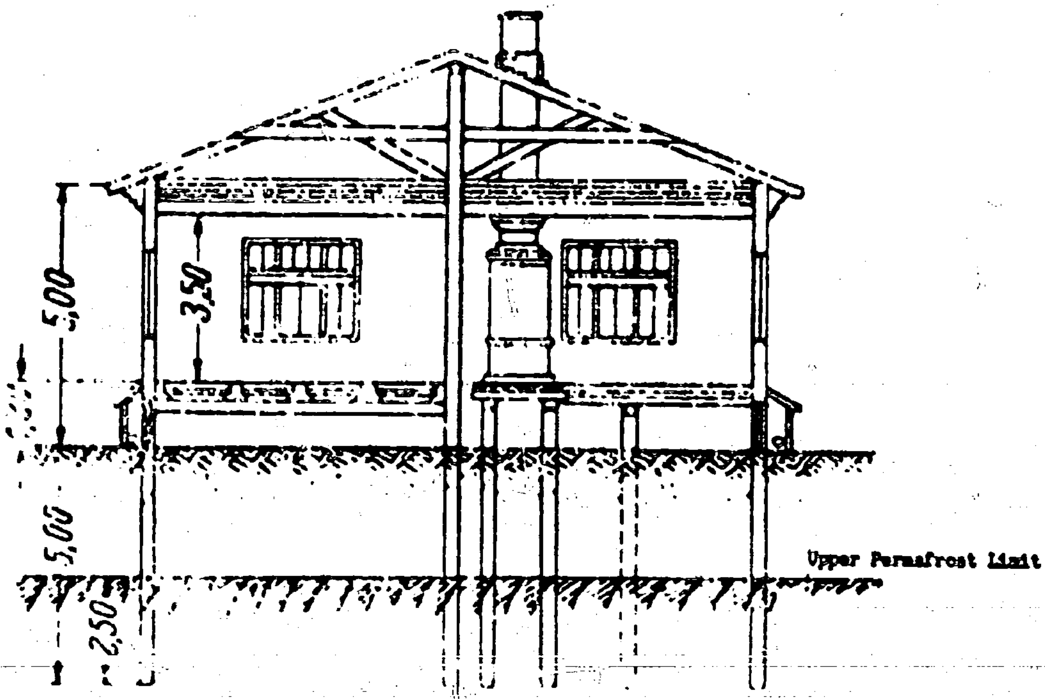


Fig. 44 - Rise of Upper Permafrost Limit Beneath Experimental Wooden Building

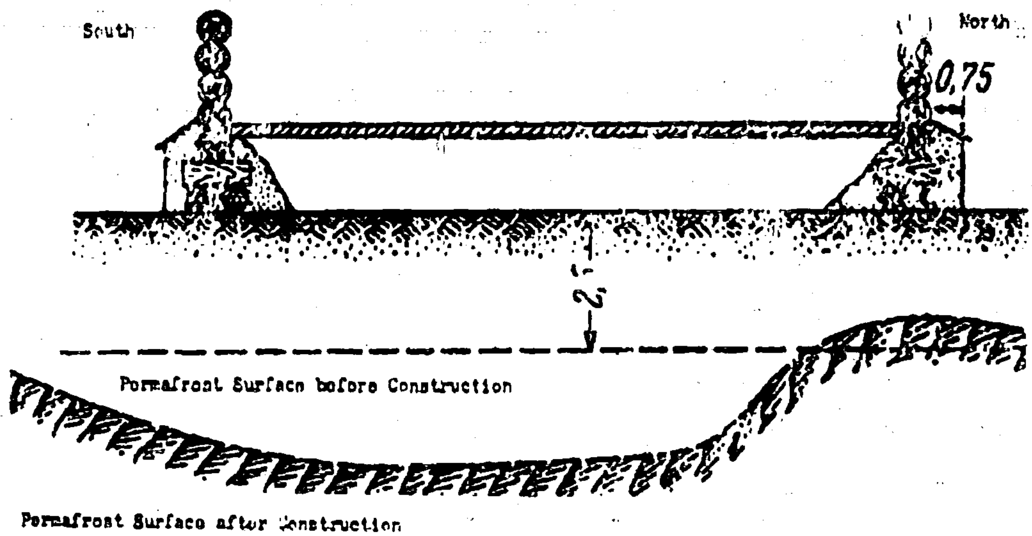


Fig. 45 - Typical Change in Upper Permafrost Limit Beneath a Building

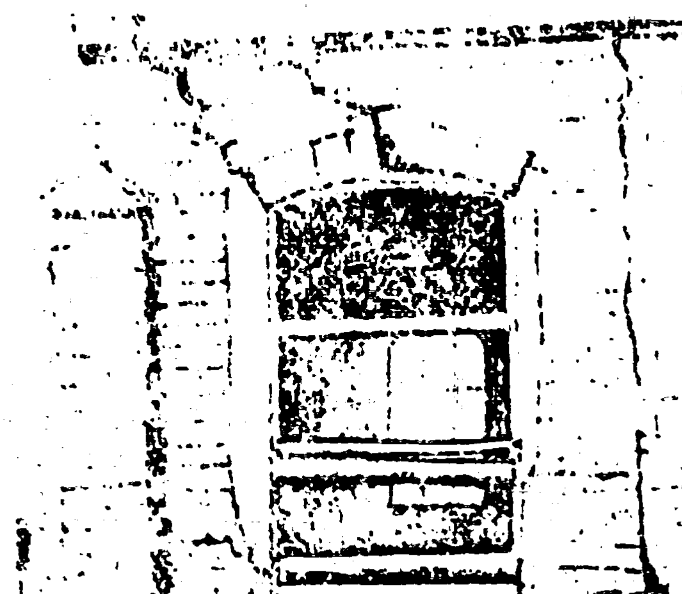


Fig. 46 - Deformation of a Depot

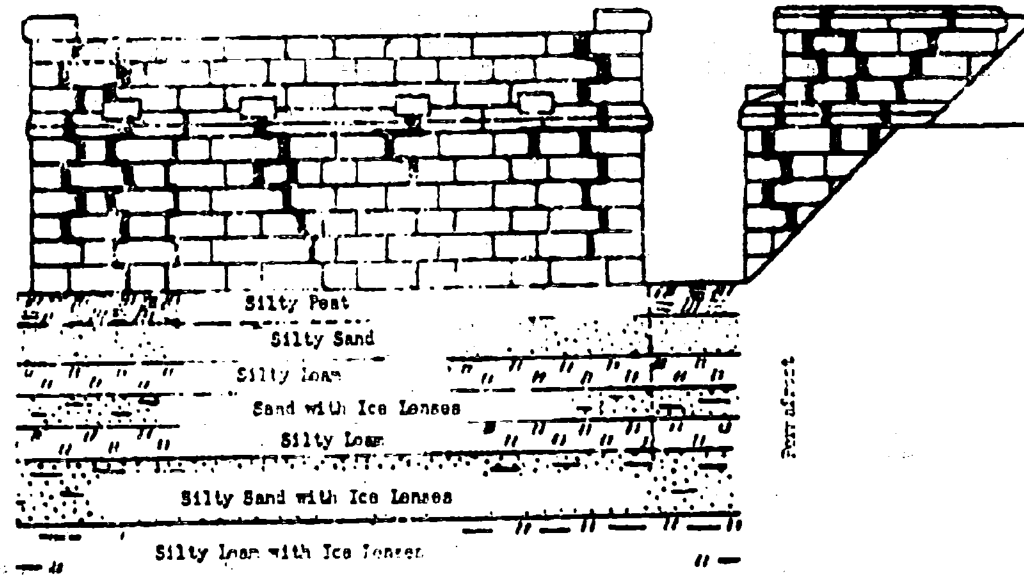


Fig. 47 - Abutment Deformation Caused by Permafrost Thawing

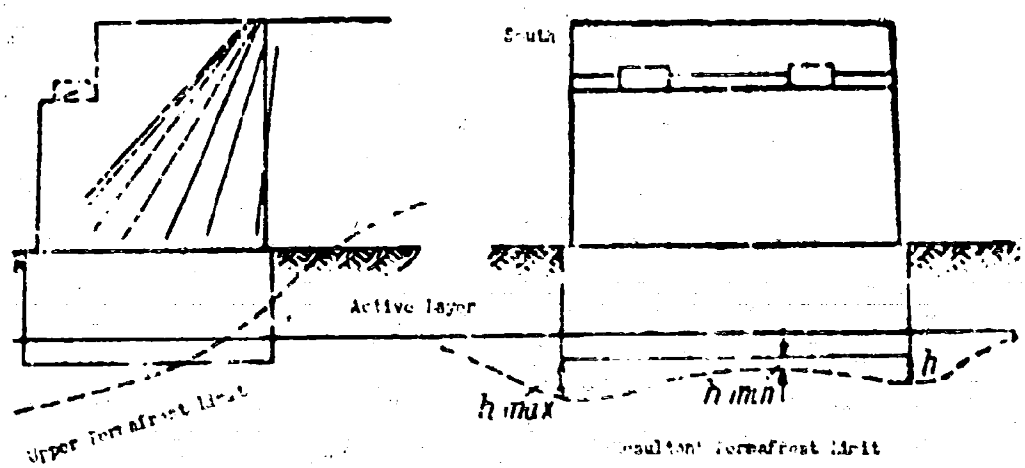


Fig. 48 - Probable Change in Permafrost Level Beneath the Abutment

Fig. 49 - Icing Formed near a House

Fig. 50 - Inhabited Houses Filled with Ice

Fig. 51 - House Wrecked by Icing

Fig. 52 - Icing Formed Within a Dismantled House

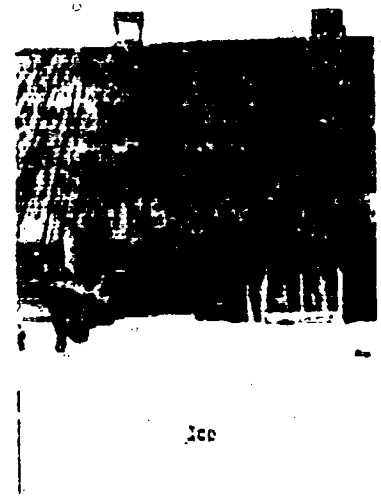


Fig. 53 - House Engulfed by Icing Covering Entire Residential Area at Skovorodino (From Elenevsky)

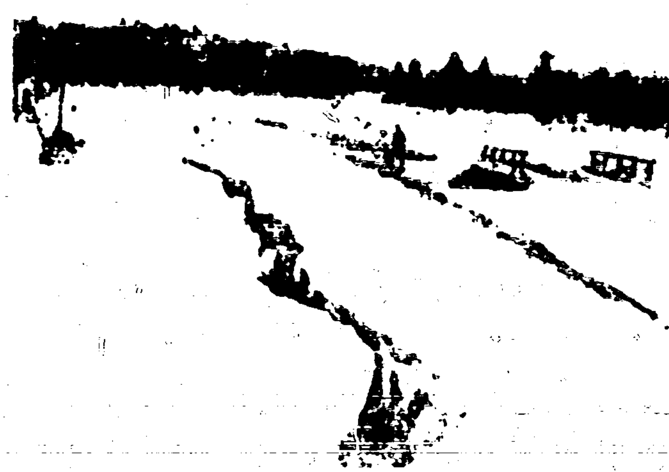


Fig. 54 - Icing Overflowing a Highway Bridge

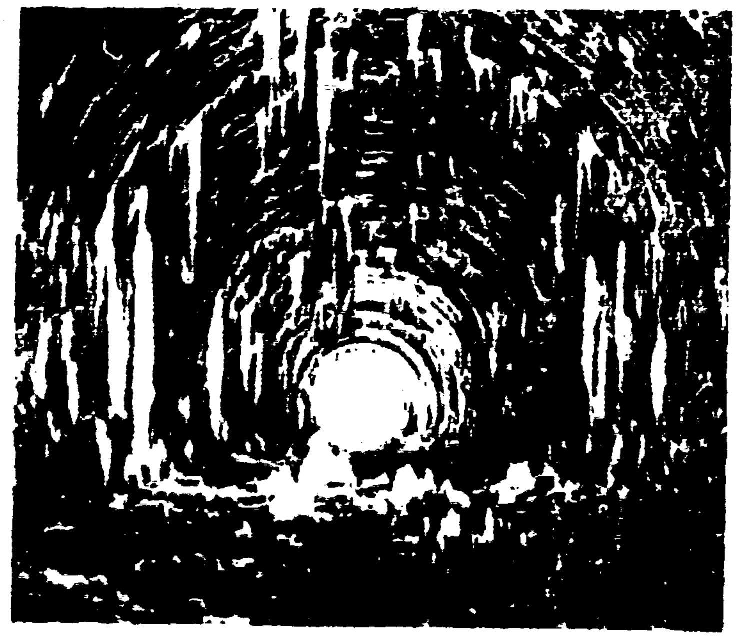


Fig. [illegible] - Icing in a Tunnel

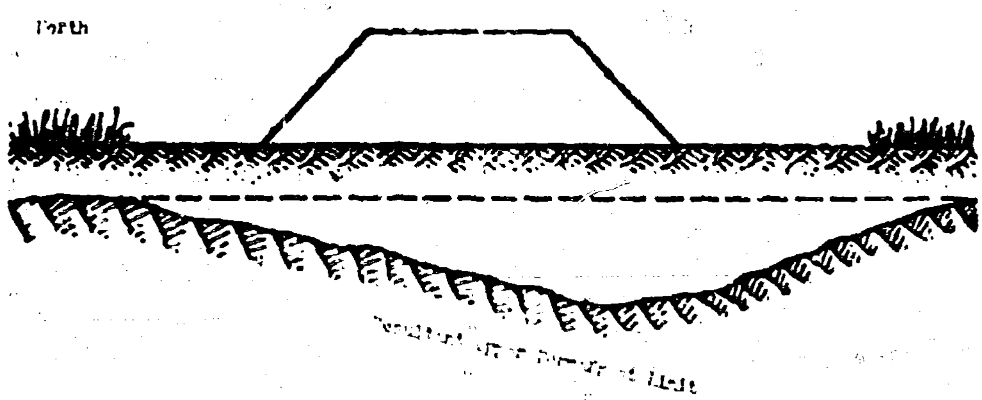


Fig. 57 - Probable Change in Position of Upper Permafrost Limit Beneath a High Fill

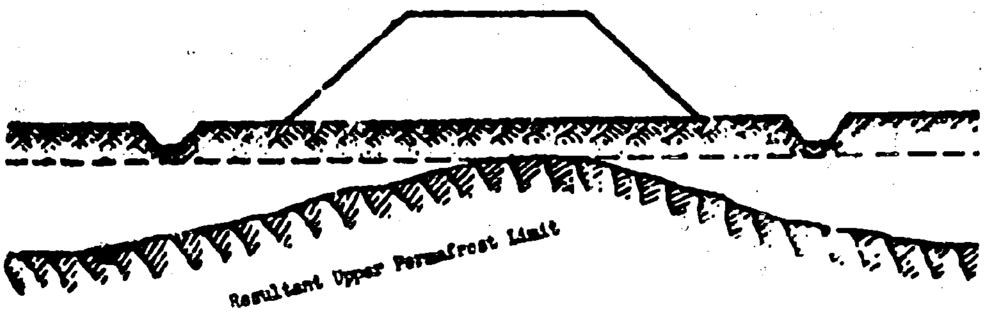


Fig. 57 - Diagram Illustrating Position of Permafrost Surface Beneath a Fill

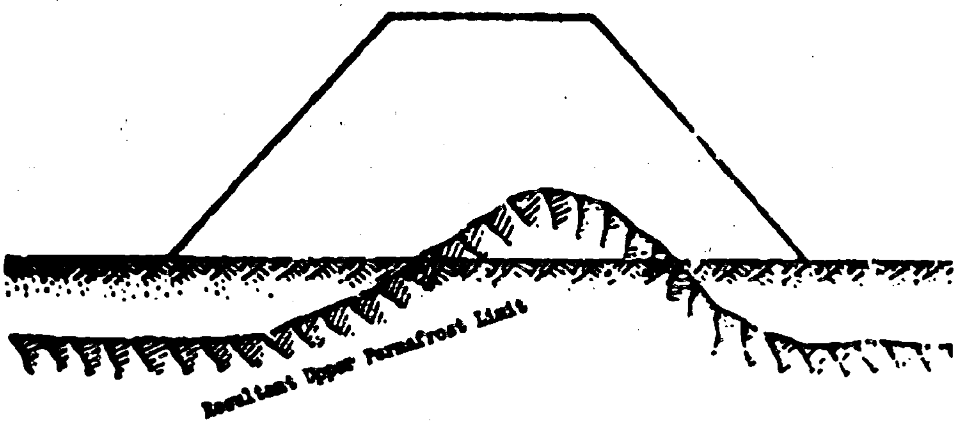


Fig. 58 - Probable Extent of Permafrost Rise into the Fill

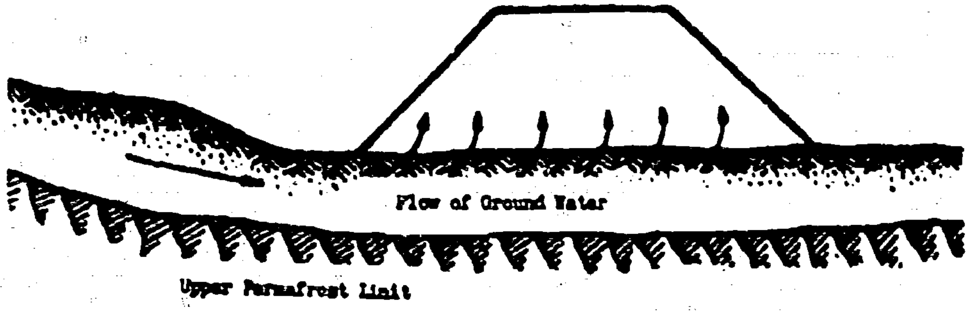


Fig. 59 - Diagram Illustrating Deformation of a Fill

Fig. 60 - Ground Icing Covering a Highway

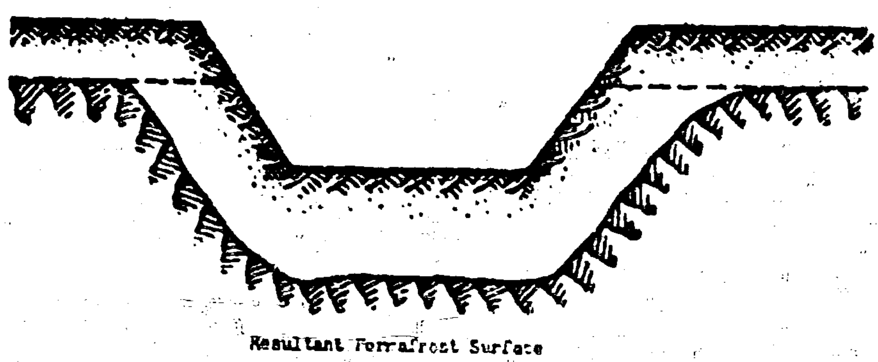


Fig. 61 - Diagram Showing Position of Upper Permafrost Limit in a Cut

Fig. 62 - Muddy Railroad Cut During Construction

Fig. 63 - Cut Filled with Slud Due to Melting of Imbedded Ice

Fig. 64 - Icefall or [illegible] a Slope of Railroad Cut

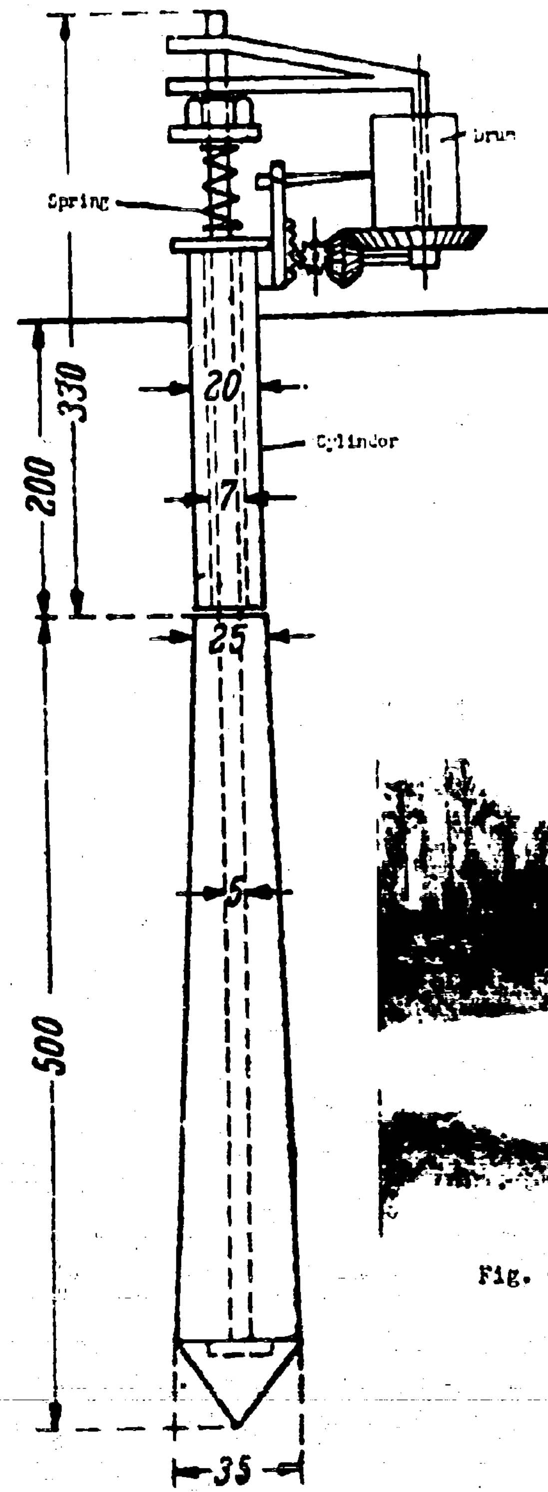


Fig. 66 - Apparatus for Measuring Heaving Forces

Fig. 65 - Melting Ice of Chichatka River for Water Supply

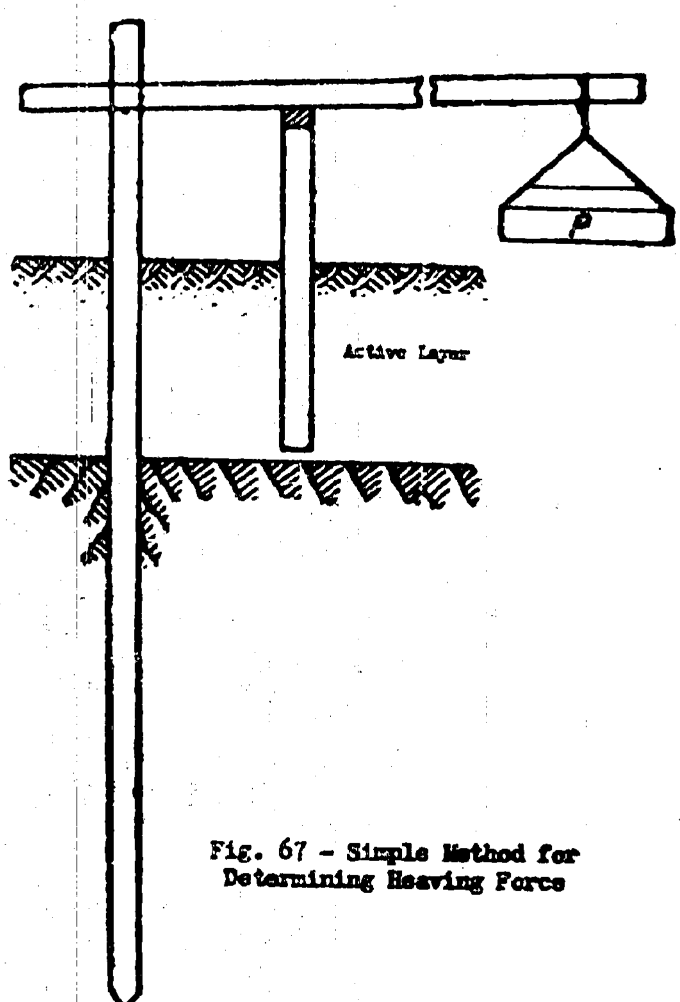


Fig. 67 - Simple Method for Determining Heaving Force

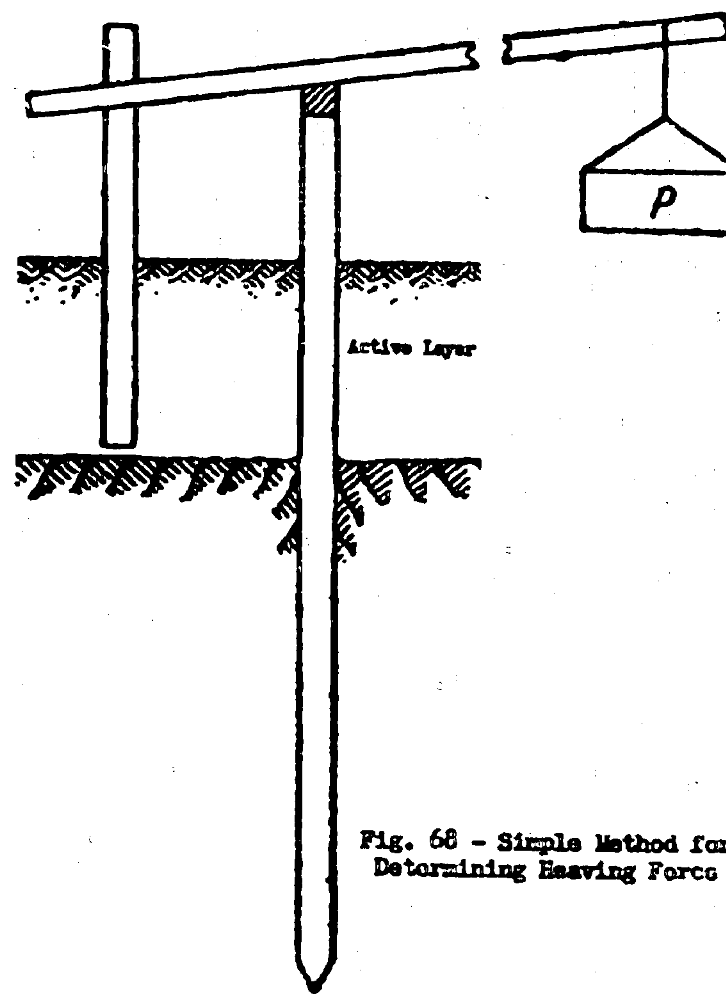


Fig. 68 - Simple Method for Determining Heaving Force

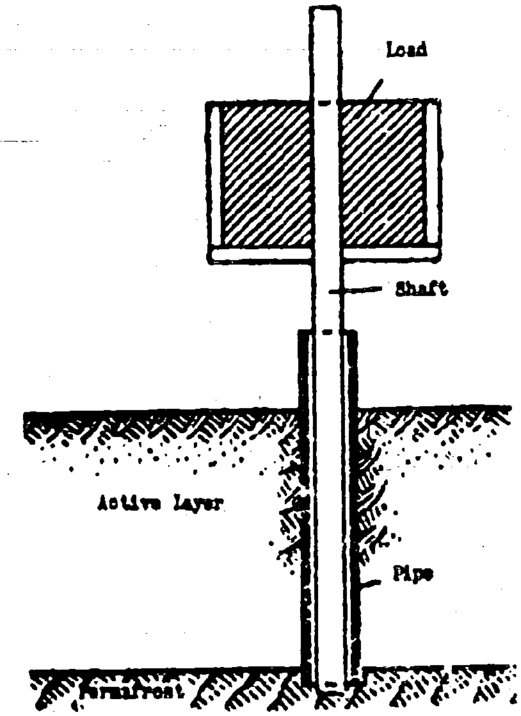


Fig. 69 - Load Test

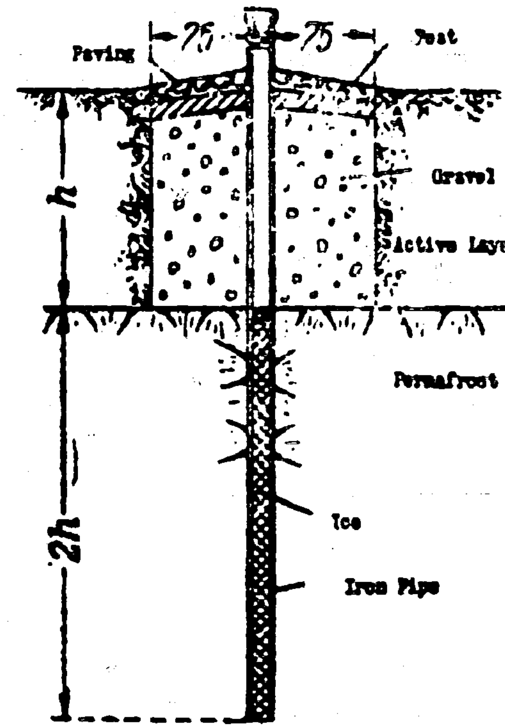


Fig. 70 - Benchmark in Permafrost

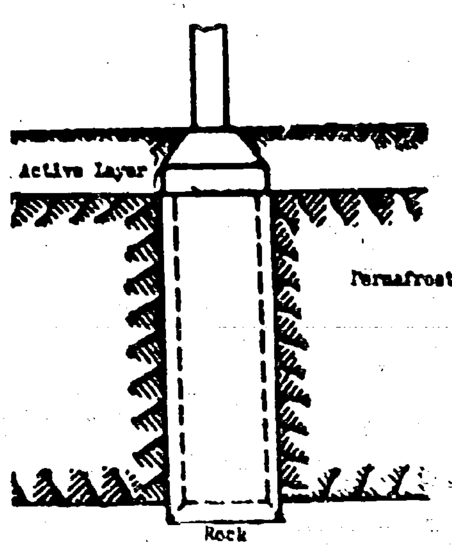


Fig. 71 - Column Foundation on Sunk Well

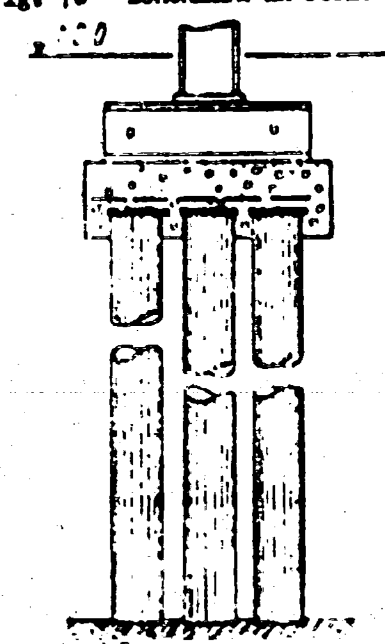


Fig. 72 - Foundation on Steel Piles

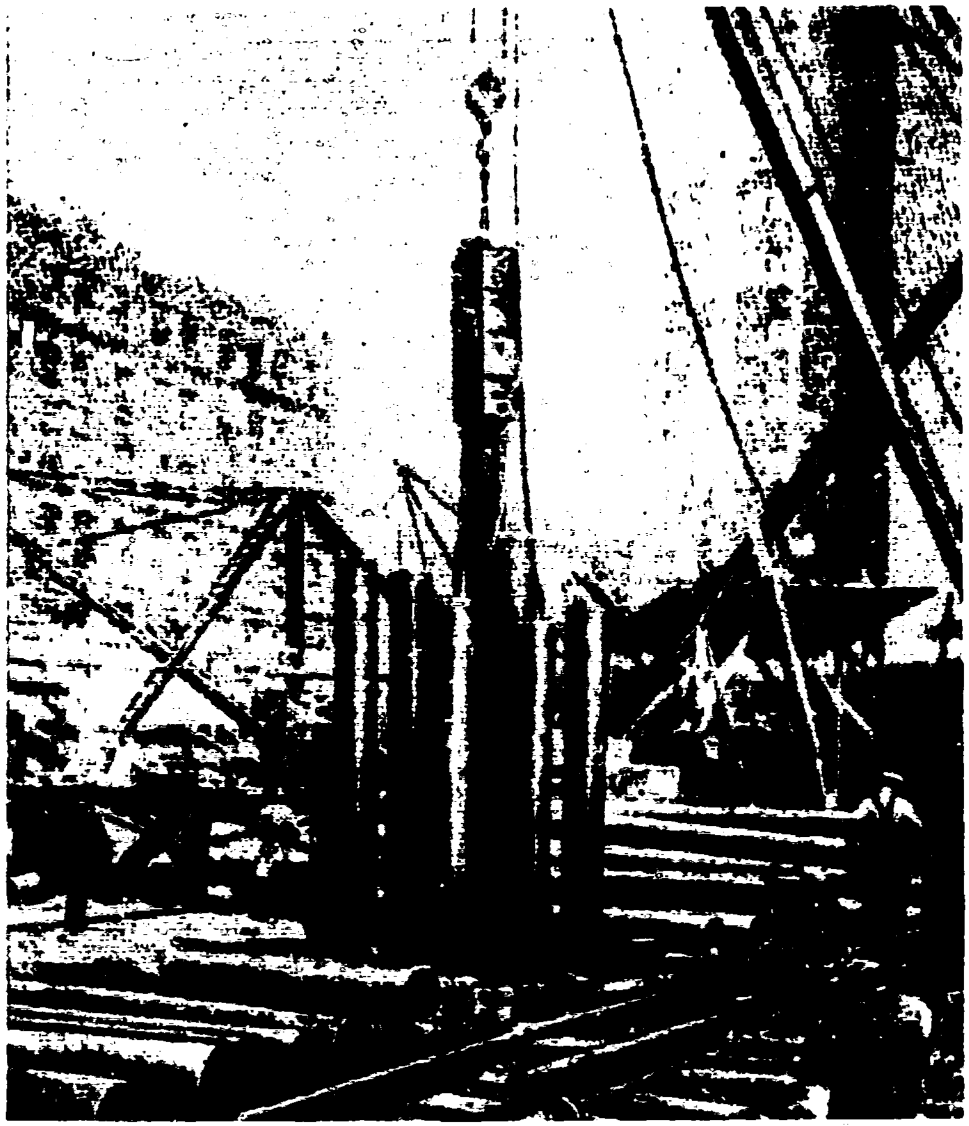


Fig. 73 - Driving Steel Piles with an American Steam Hammer

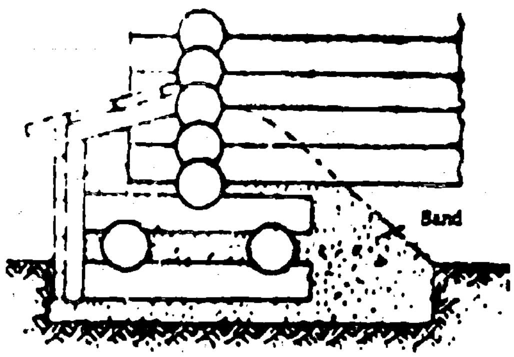


Fig. 74 - Wooden Building on Cribs

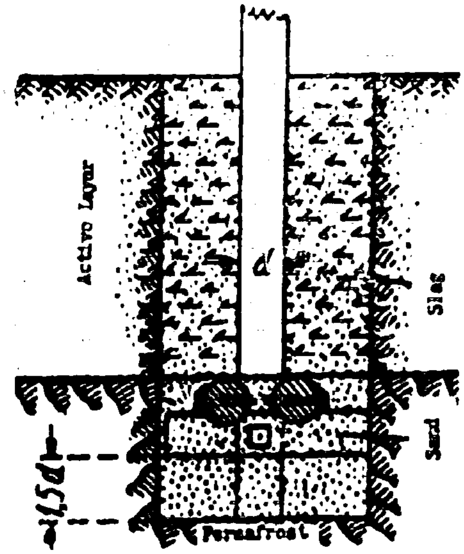


Fig. 75 - Anchoring a Wooden Column

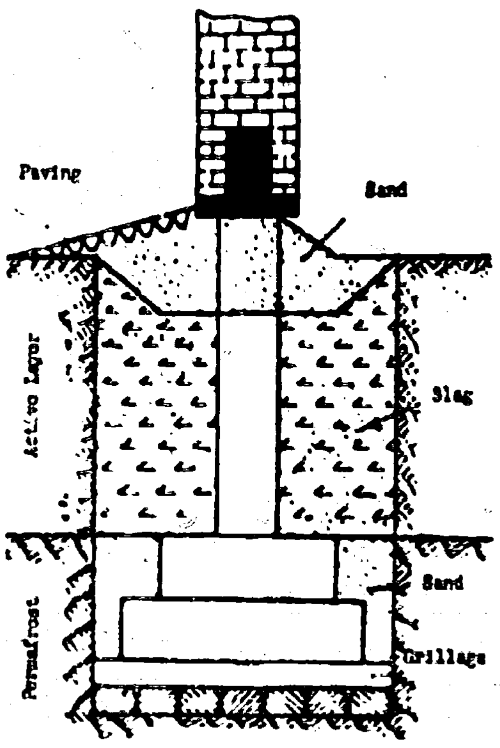


Fig. 76 - Anchoring a Reinforced Concrete Column

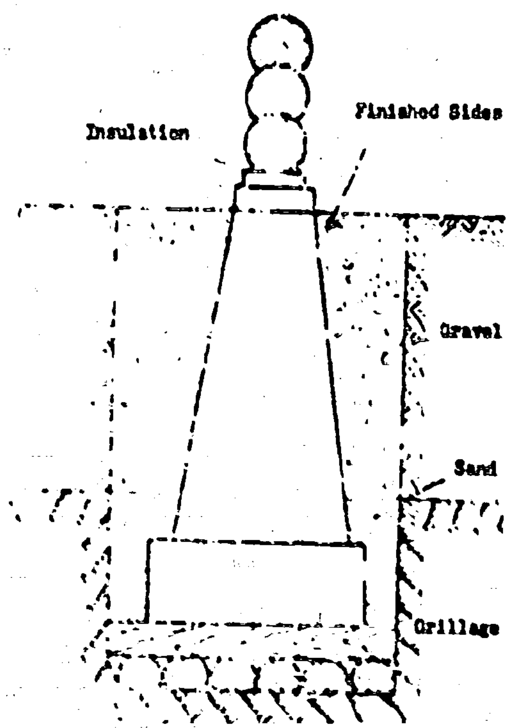


Fig. 77 - Rubble Foundation

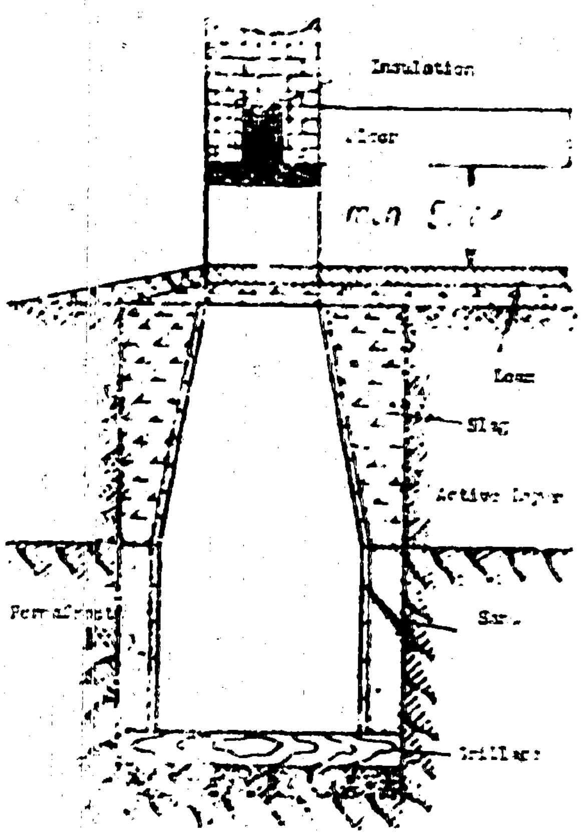


Fig. 78 - Heated Building on Rubble Concrete Foundation

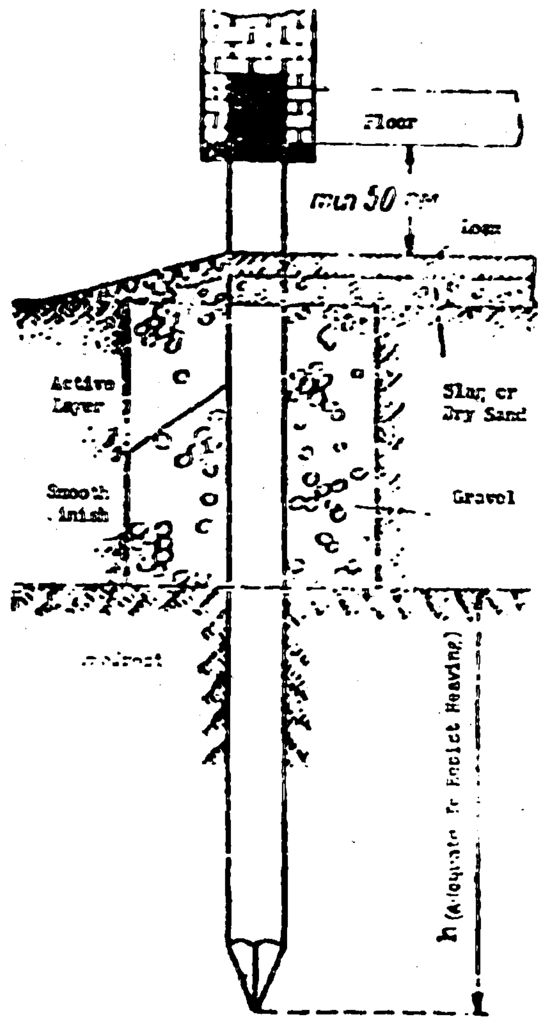


Fig. 79 - Pile Foundation

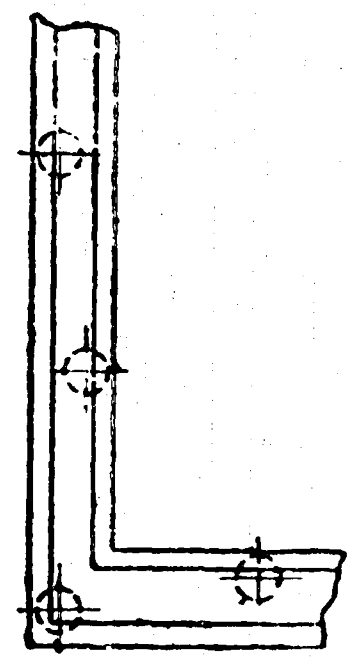
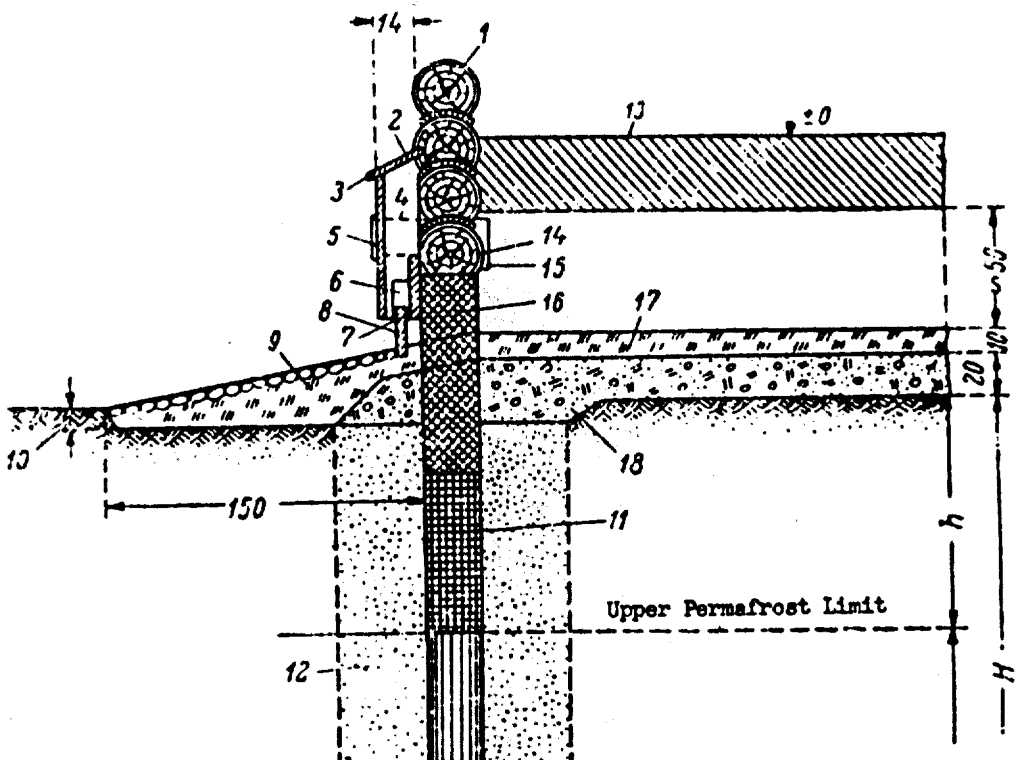


Fig. 80 - Staggered Arrangement of Piles



1. Outer wall. 2. Eaves of 25 cm board dipped in a 3 percent sodium fluoride solution. 3. Quarter round 2.5 cm thick. 4. 16 by 16 cm creosoted beam spaced at 70 cm intervals. 5. 20 by 30 cm air vents every 1.5 to 2.0 m. 6. Air space. 7. Creosoted joist. 8. Creosoted board measuring 5 by 20 cm. 9. Clay or loam covered with tamped rubble. 10. Depth 10 to 20 cm. 11. Two layers of tar paper over a layer of tar. 12. Solidly tamped backfill of excavated material. 13. Suitable roof. 14. Lower three timbers creosoted and covered with special putty. 15. Bottom half of lower timber covered with creosoted felt overlain by tar paper. 16. Jacket. 17. Tamped clay loam. 17. Tamped moss cushion. 19. Anchor.

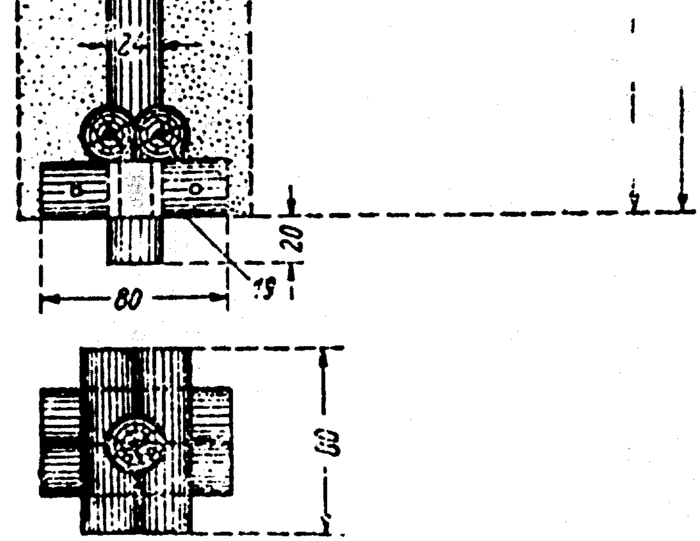
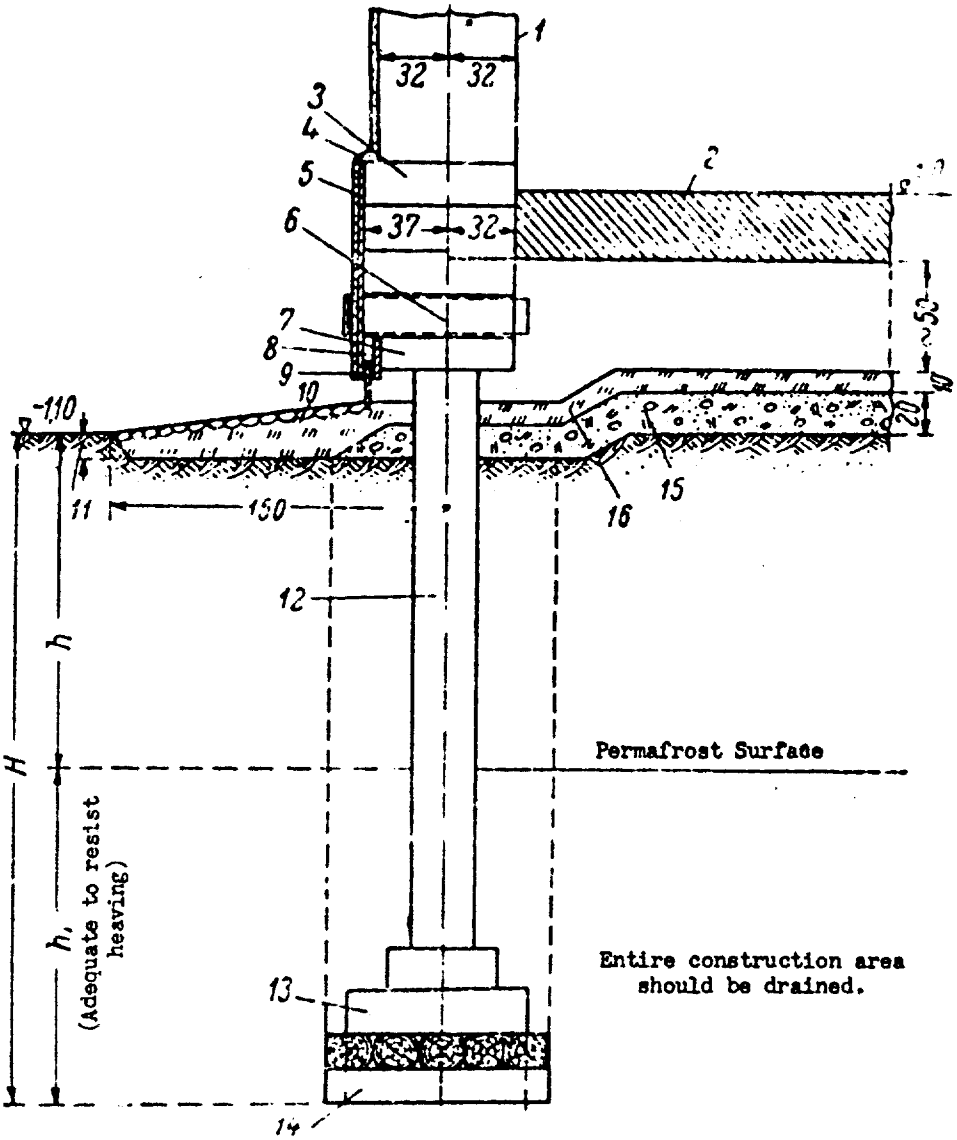


Fig. 81 - Timber Column Foundation Designed by V. A. Byalinitsky



1. Outer wall. 2. Suitable floor. 3. Slag concrete masonry. 4. Plaster 2.5 cm thick. 5. Siding boards 2.5 cm thick. 6. 20- by 30-cm air vents at intervals of 2.0 to 2.5 m. 7. Reinforced concrete beam. 8. 2.5-cm board. 9. Creosoted board 4 cm thick. 10. Sandy or clay loam covered with tamped rubble. 11. Depth 10 to 20 cm, depending on thickness of vegetation cover. 12. Reinforced concrete column of design cross section. 13. Reinforced concrete slab of design cross section. 14. Insulating timber platform of 16-cm logs. 15. Insulating layer of tamped moss. 16. Well-tamped sandy loam or clay loam.

Fig. 92 - Reinforced Concrete Column Foundation (Design by Byalinitsky)

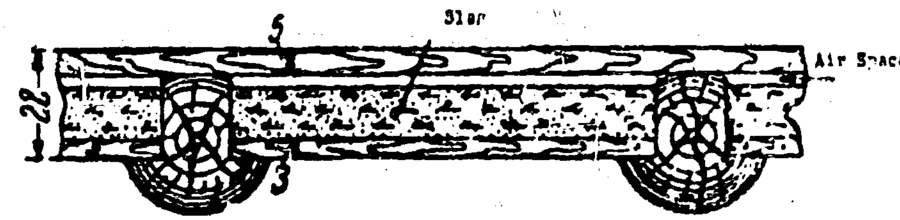


Fig. 83 - Improper Design of Wooden Floor

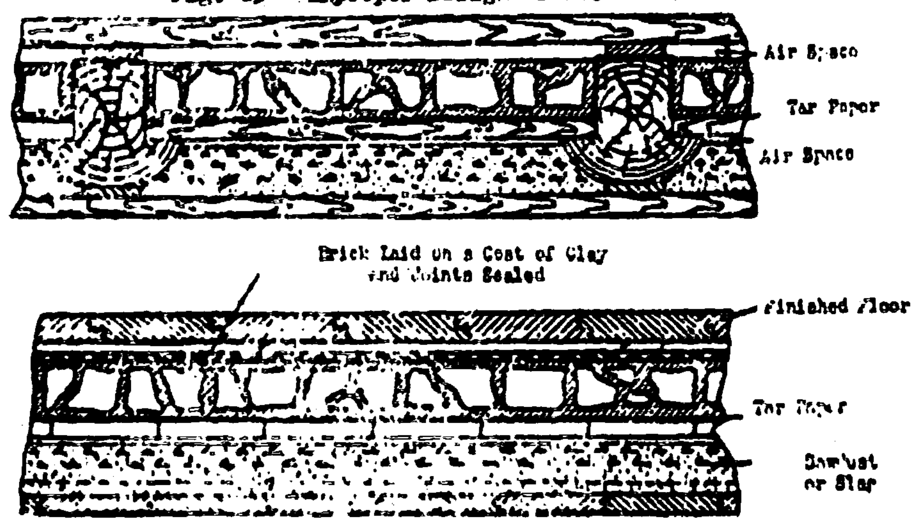


Fig. 84 - Bikov's Design of Wooden Floor

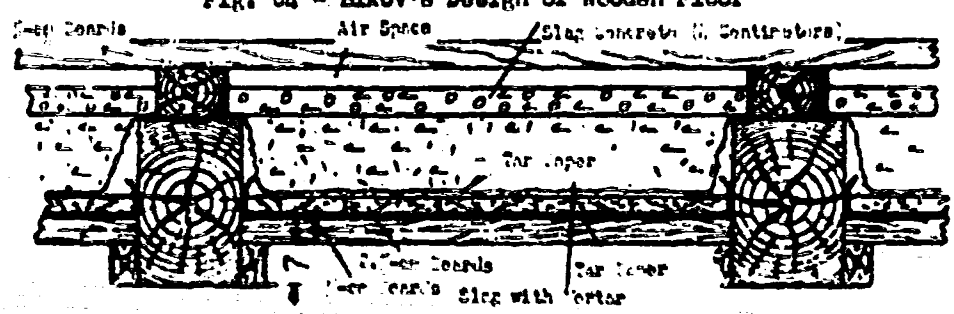


Fig. 85 - Proper Design of Wooden Floor

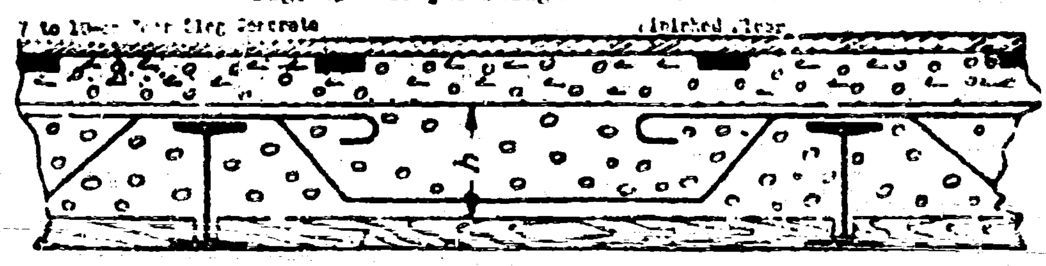


Fig. 86 - Slag Concrete Floor

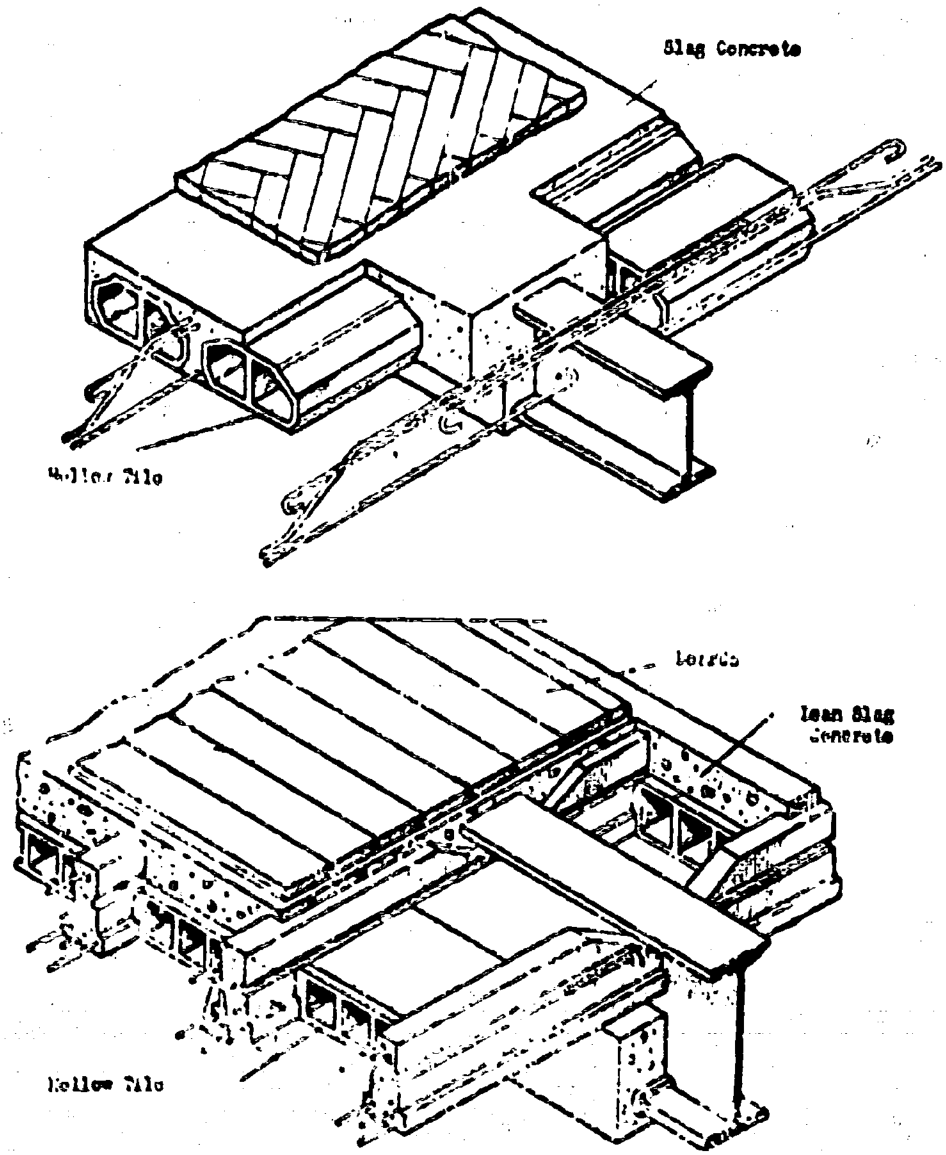


Fig. 87 - Typical Hollow Tile Floors

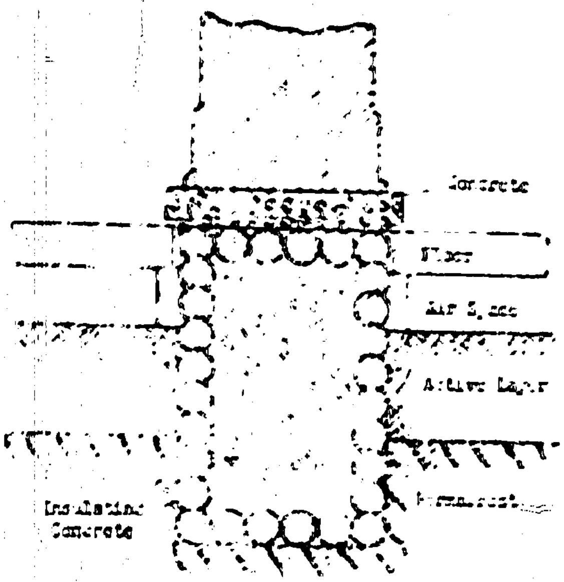


Fig. 88 – Crib Foundation for a Stove

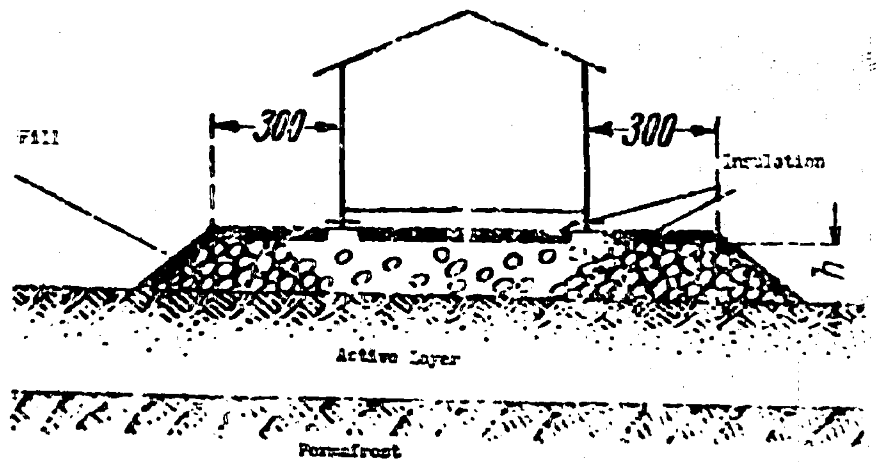


Fig. 89 – Wooden Building on a Fill

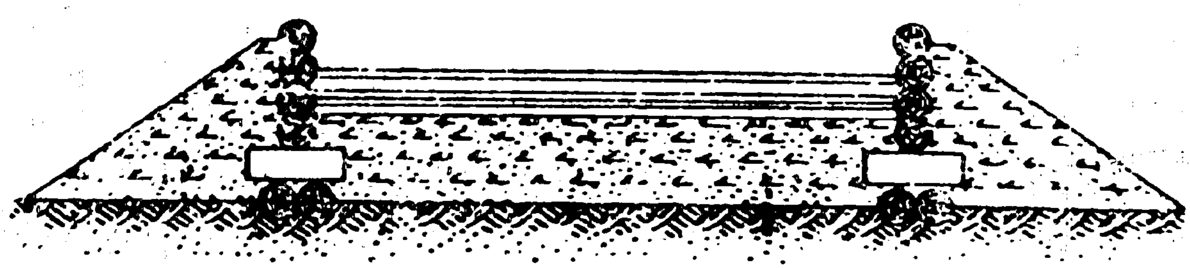


Fig. 90 – Improper Design of Foundation Fill

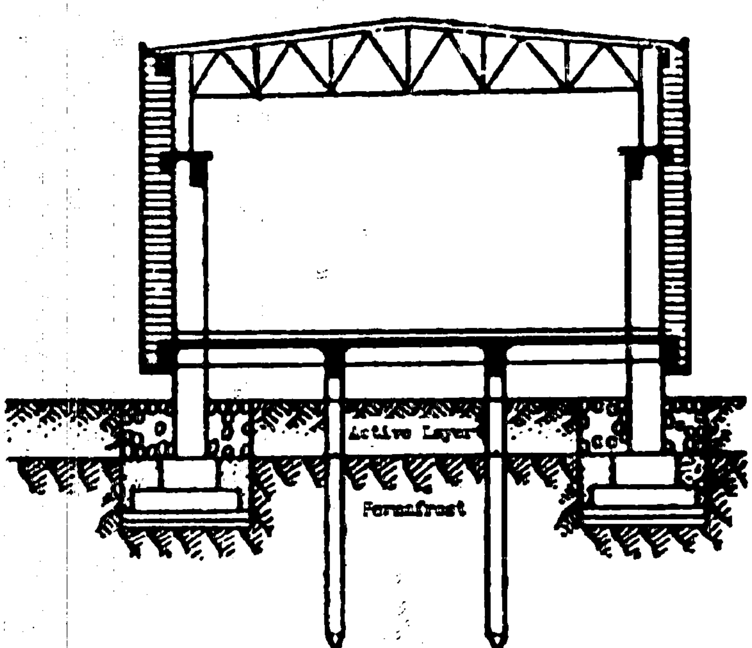


Fig. 91 - Foundation Design for Heated Industrial Buildings

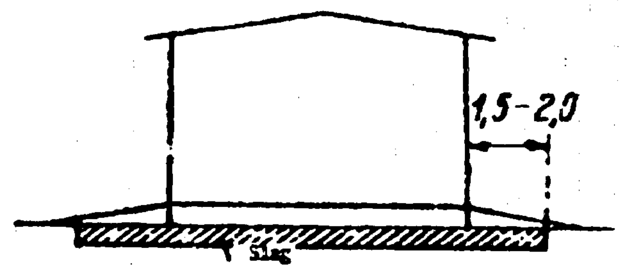


Fig. 93 - Design Utilizing Slag Cushion

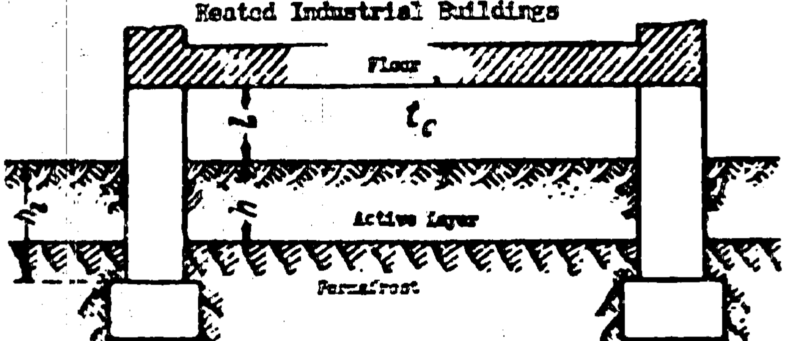


Fig. 92 - Variables in Air Space Design

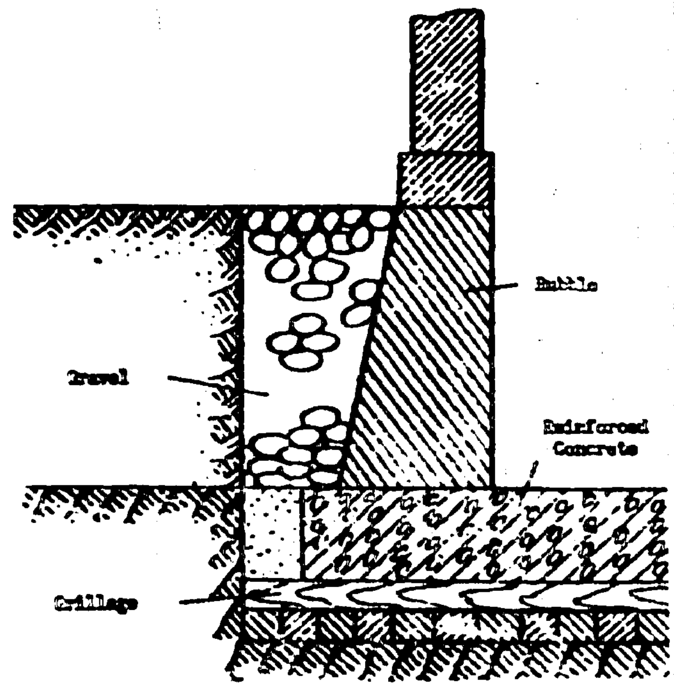


Fig. 94 - Water Tower Foundation on Permafrost

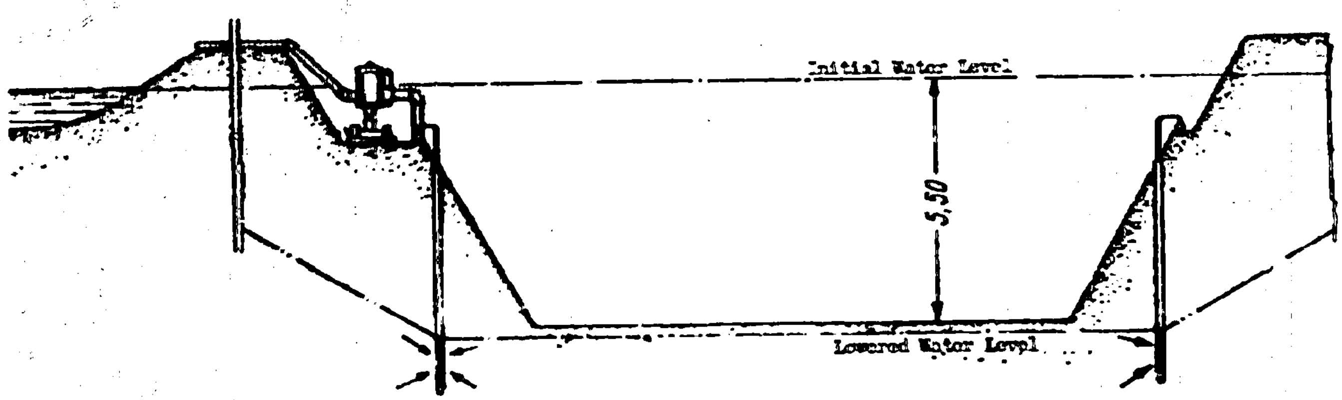
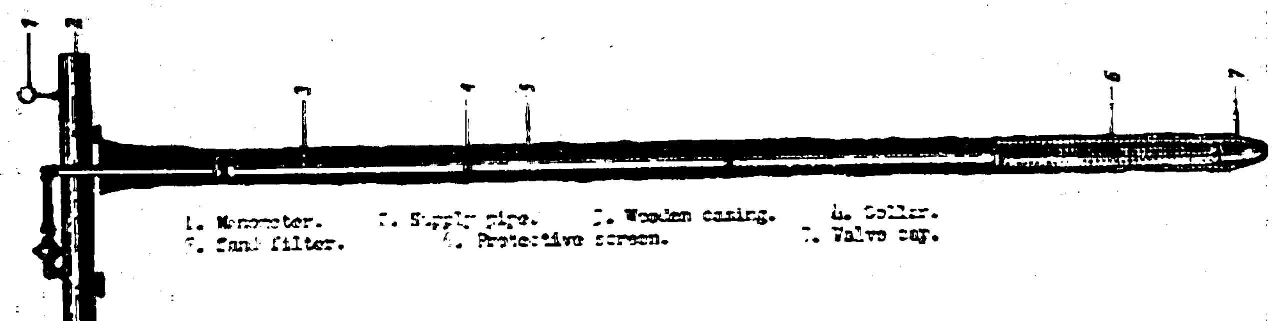


Fig. 95 – The Moretrench Wellpoint System



1. Manometer. 2. Supply pipe. 3. Wooden casing. 4. Collar.
5. Sand filter. 6. Protective screen. 7. Valve cap.

Fig. 96 – The Moretrench Wellpoint

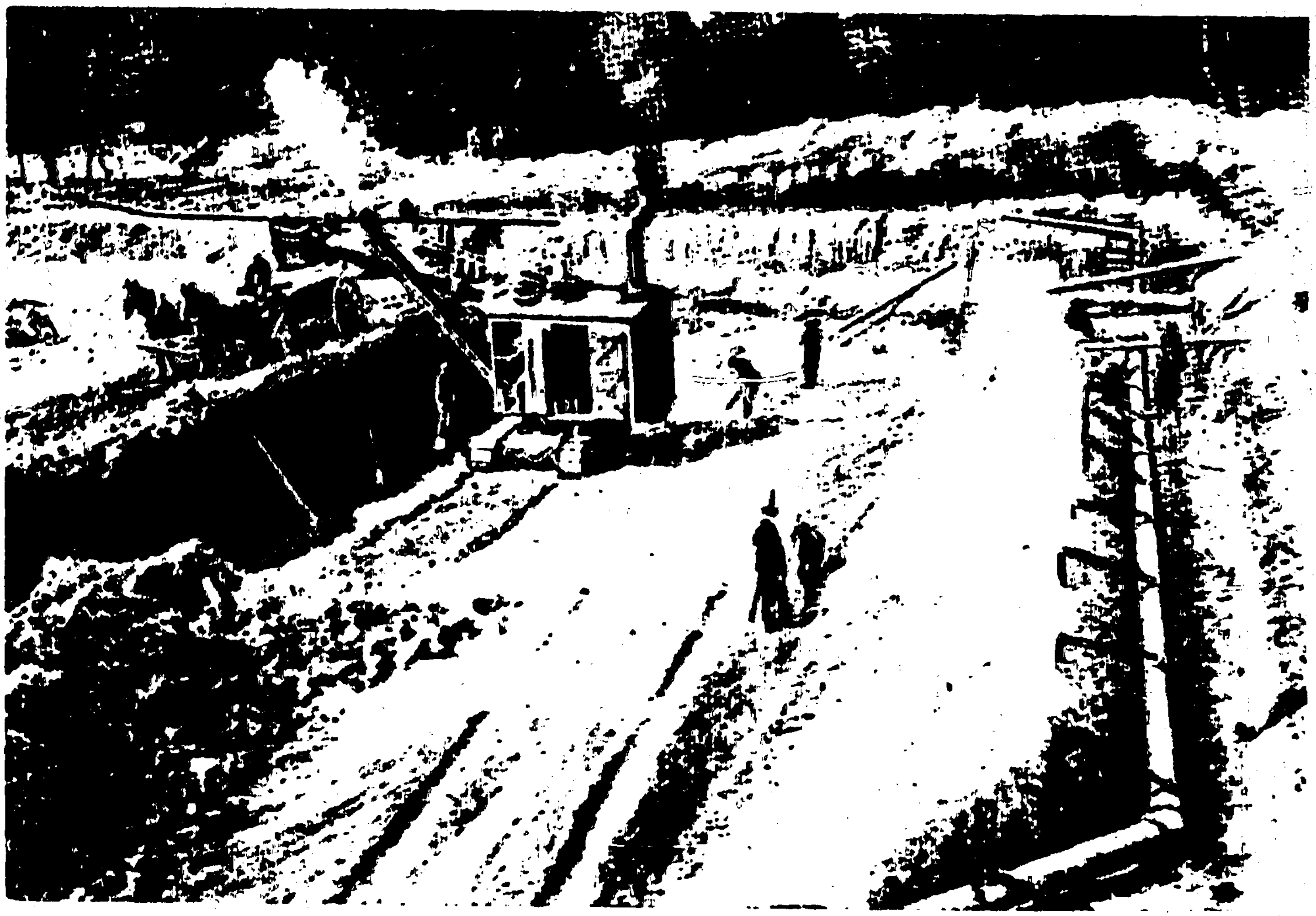


Fig. 97 - Excavation Dried by Lowering the Ground-Water Level

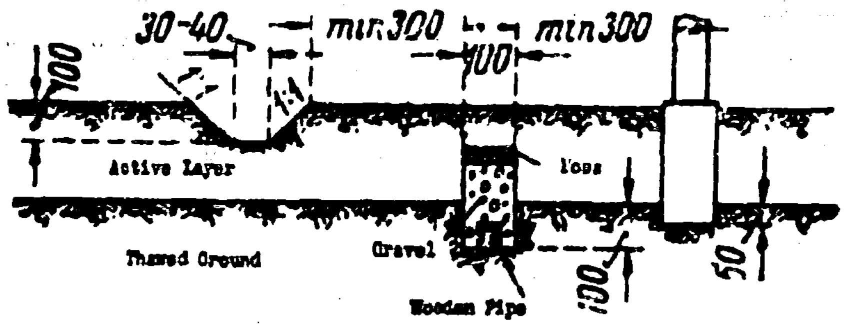


Fig. 98 - Drainage Installations near a Building

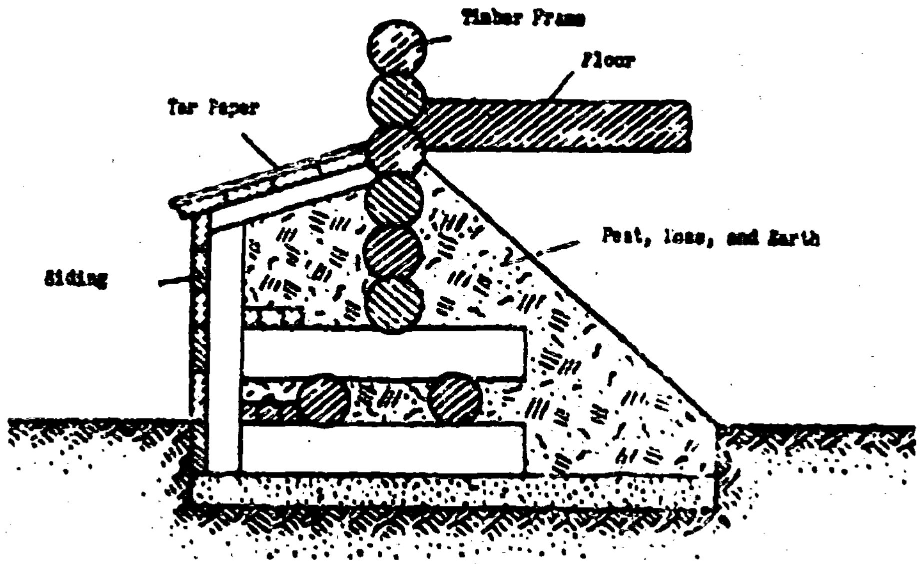


Fig. 99 - Wooden Building on Crib Foundation with Insulated Air Space

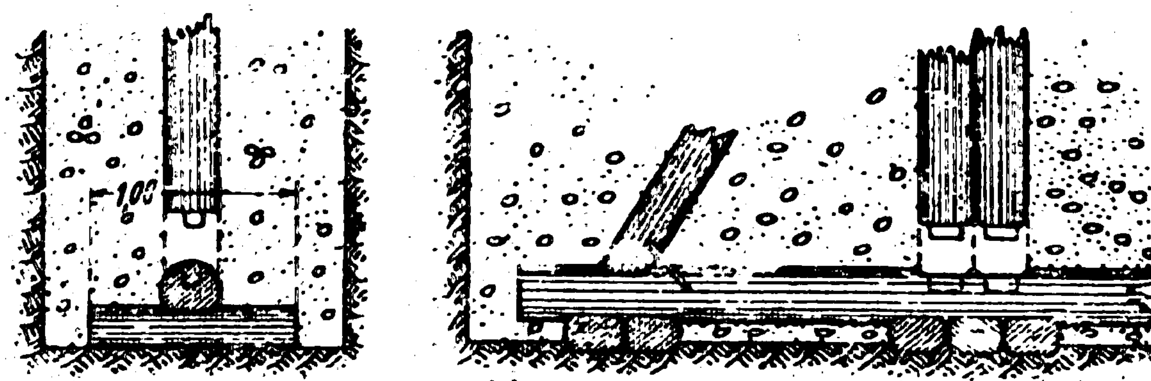


Fig. 32 - Columns Separated from Grill by Heaving

Fig. 33 - Failure of Pile Splices Due to Heaving

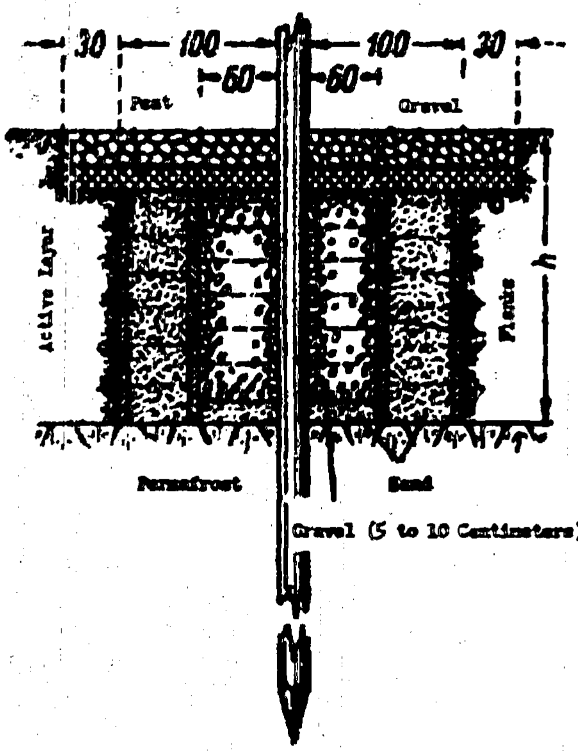


Fig. 103 - Backfill Around a Pile in Excessively Wet Silty Active Layer

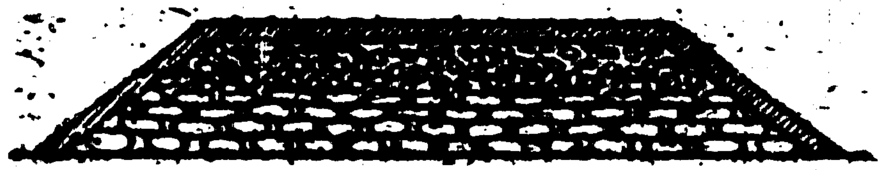


Fig. 104 - Proper Design of Filter Dam

Fig. 105 - Improper Design of Filter Dam

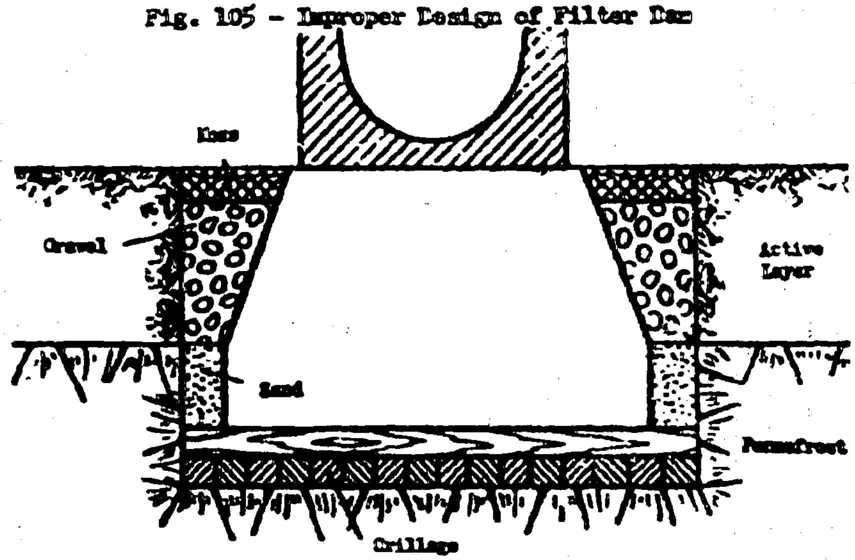


Fig. 106 - End Pier for a Culvert in Active Layer Susceptible to Swelling

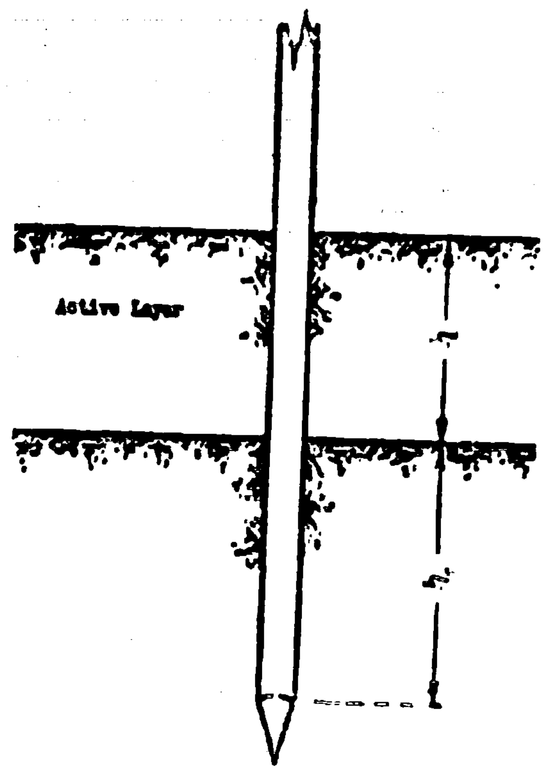


Fig. 107 - Pile Depth in Talik

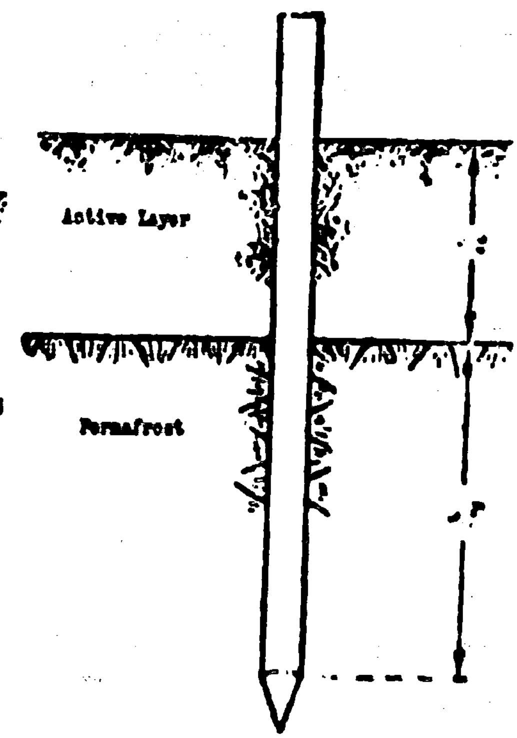


Fig. 108 - Pile Depth in Permafrost

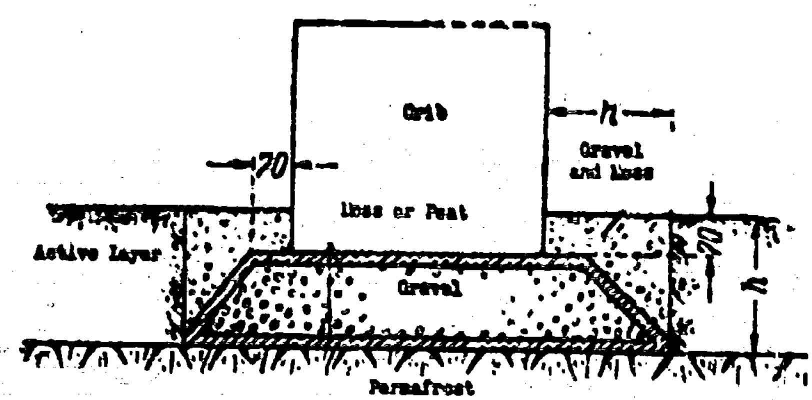


Fig. 109 - Crib Foundation

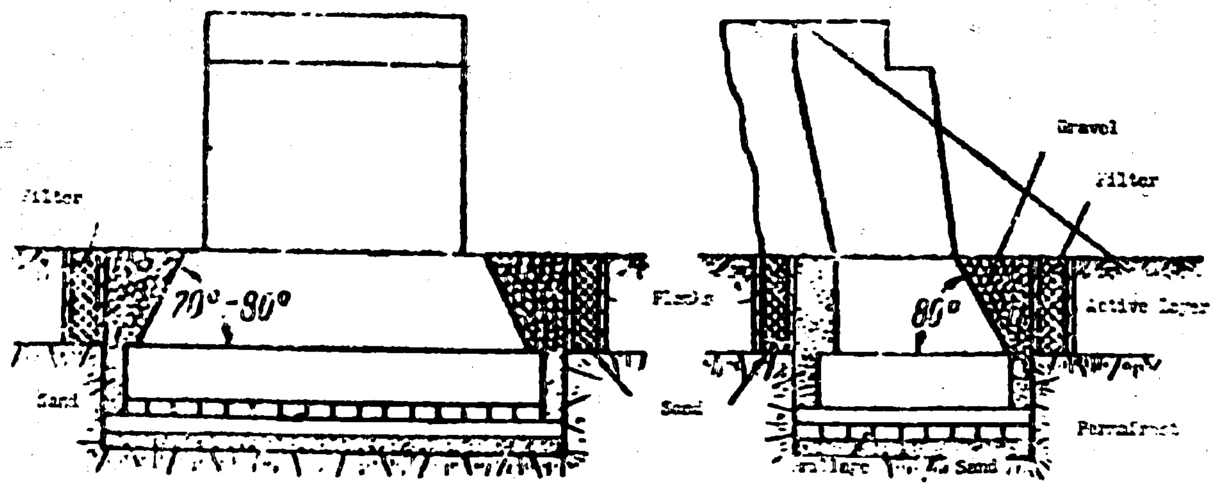


Fig. 110 – Cushioned Abutment

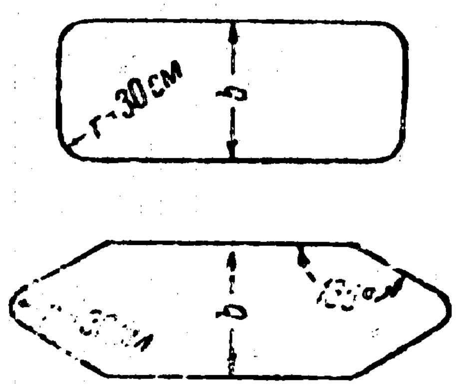


Fig. 111 – Suitable Pier Shapes

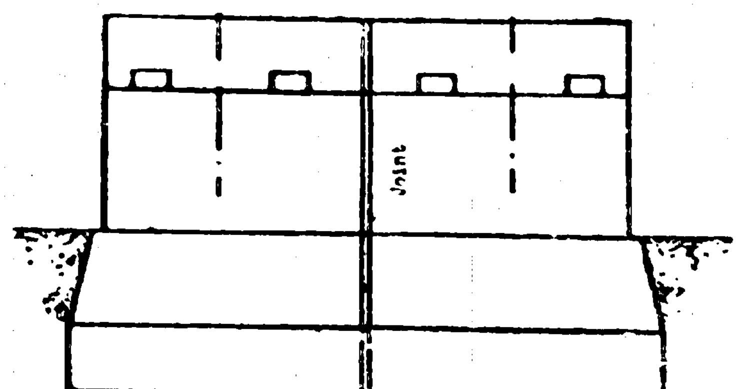


Fig. 112 – Abutment for Double Track Bridge

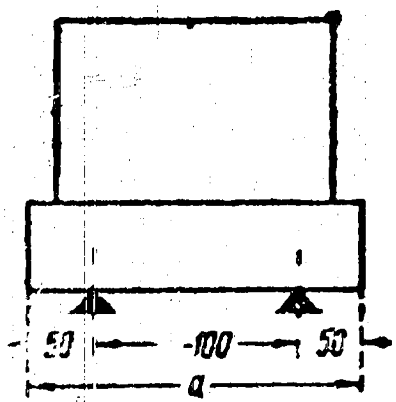


Fig. 113 - Simply Supported Beam

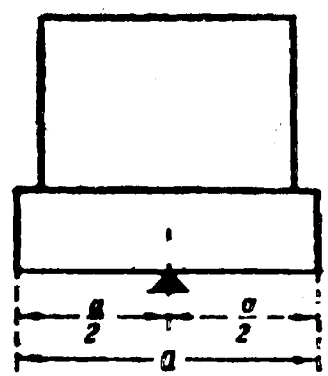


Fig. 114 - Double Cantilever Beam

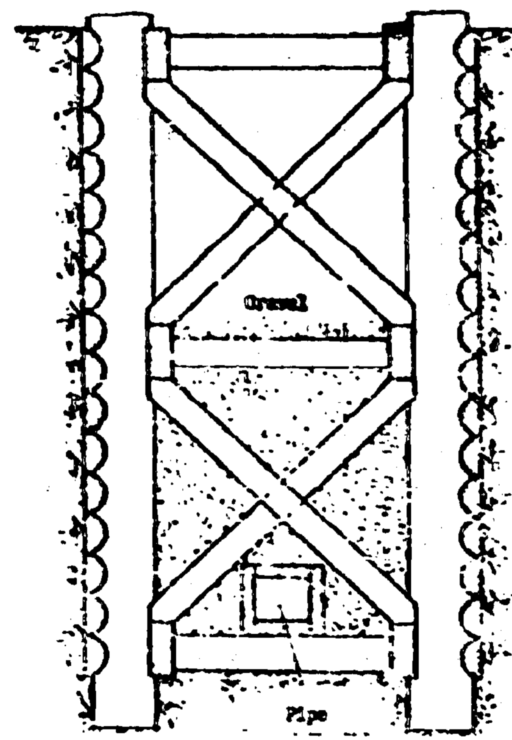


Fig. 115 - Braced Wooden Trough

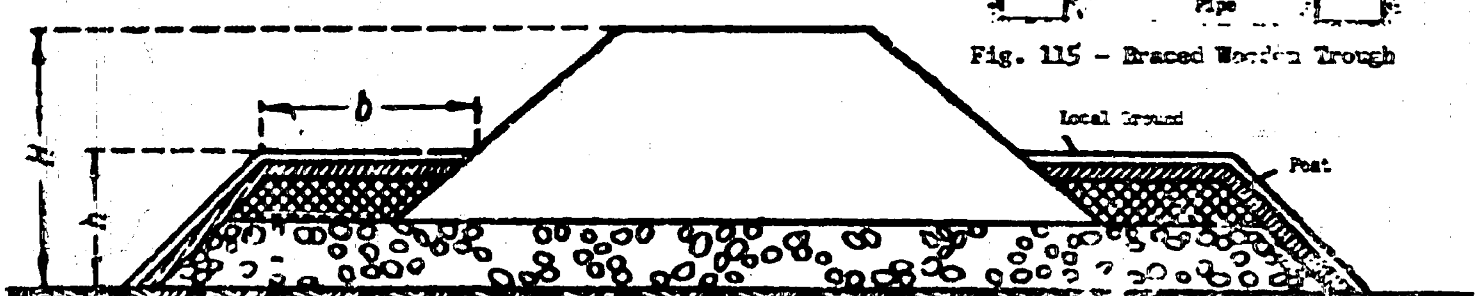


Fig. 116 - Fill on a Special Base of Well-Draining Material

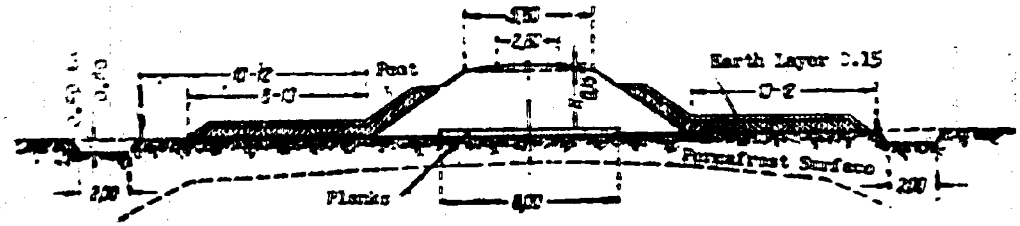


Fig. 117 - Fill of Coarse Material 2 to 3 Meters High Erected on a Bog (Design by A. Kurtinov)

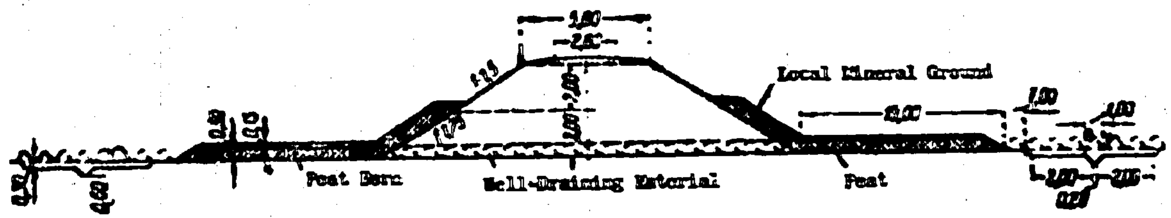


Fig. 118 - Fill of Silty Material 2 to 5 Meters High Erected on a Hummocky Bog (Design by A. Kurtinov)

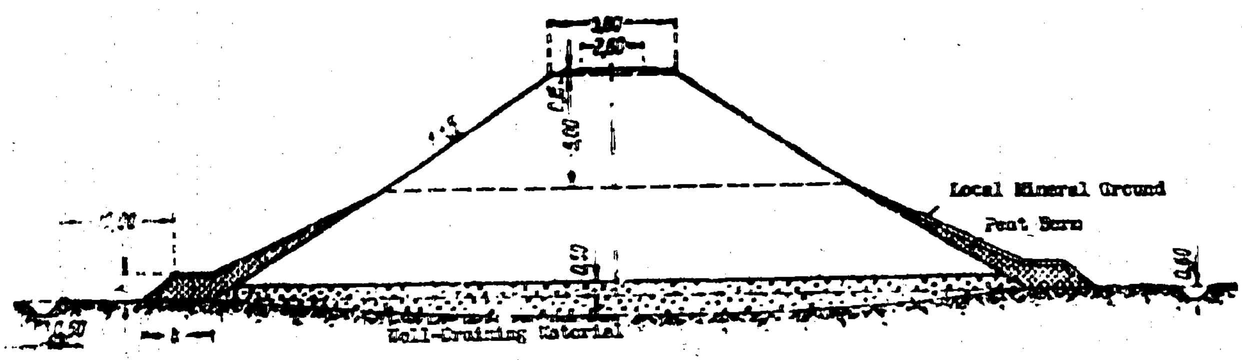


Fig. 119 - High Fill of Fine Sand and Loam Erected on a Peat Bog (Design by A. Kurtinov)

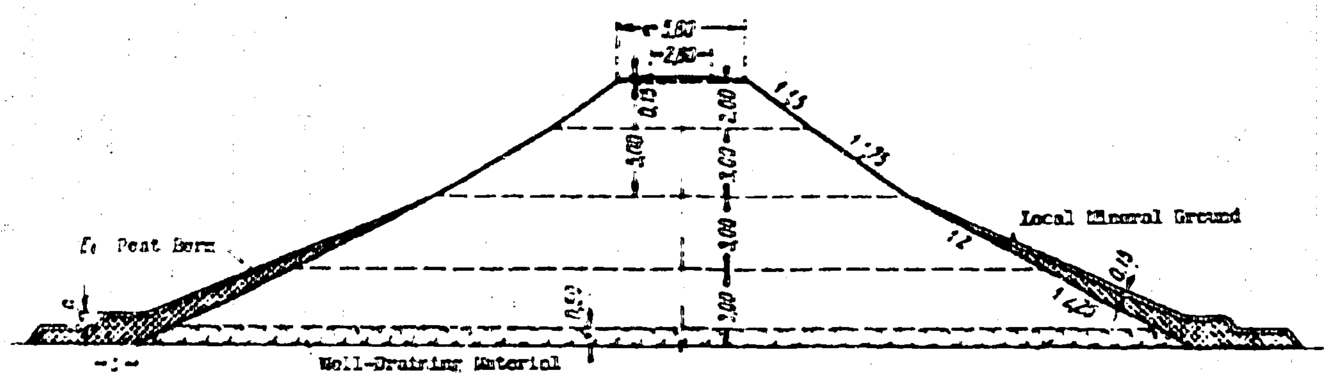


Fig. 120 - High Fill of Silty Material on a Hummocky Bog (Design by A. Kurtinov)

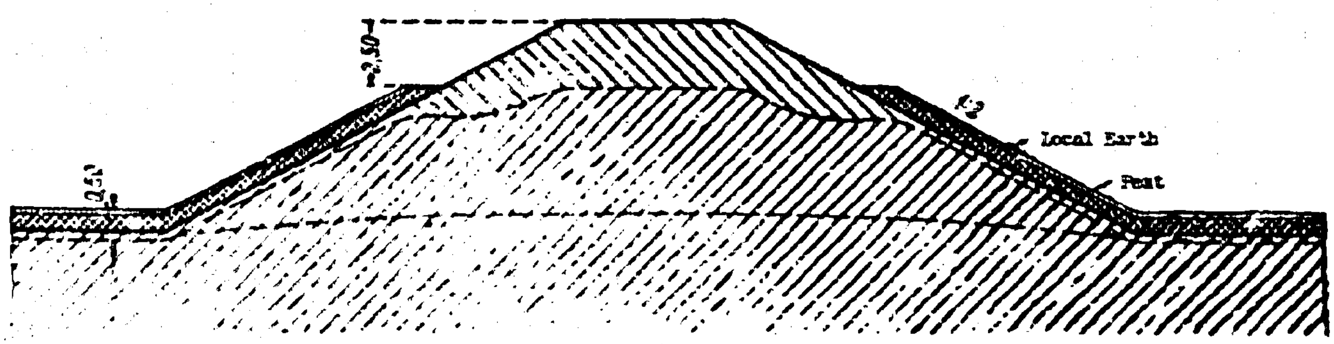


Fig. 121 - Fill of Fine-Textured Material (Design by E. I. Sukhodolsky)

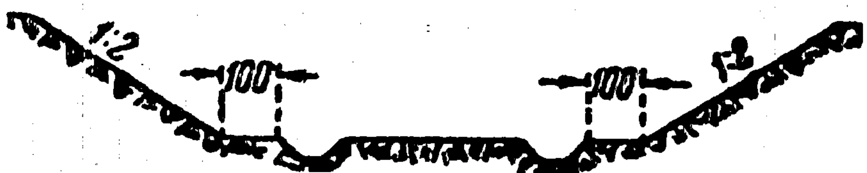


Fig. 122 - Optimum Profile for a Cut in Permafrozen Fine Sand, Loamy Sand, or Clay Loam

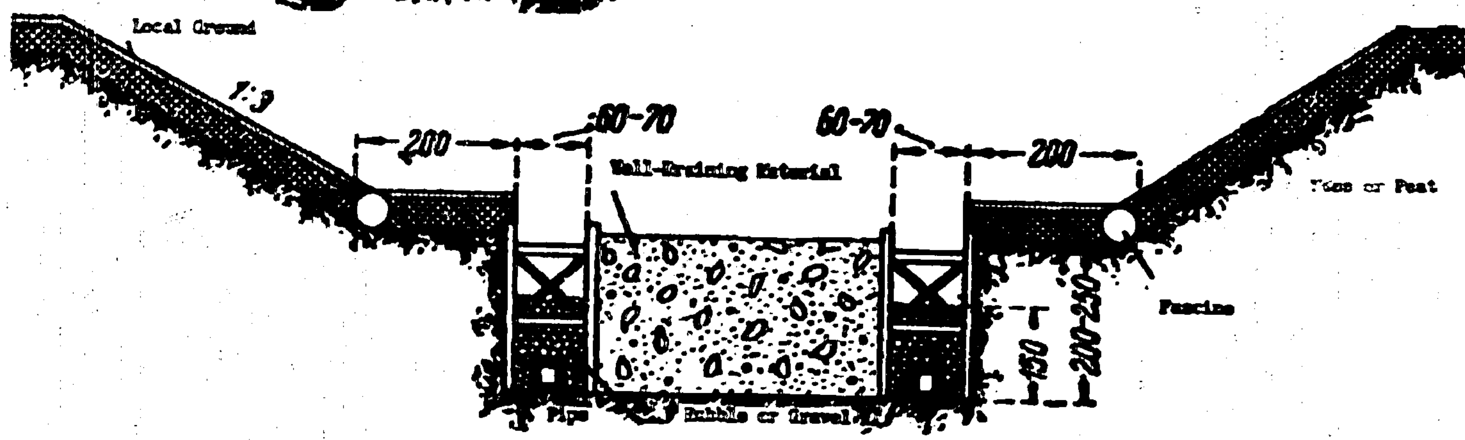


Fig. 123 - Design of a Cut in Permafrozen Wet Silt

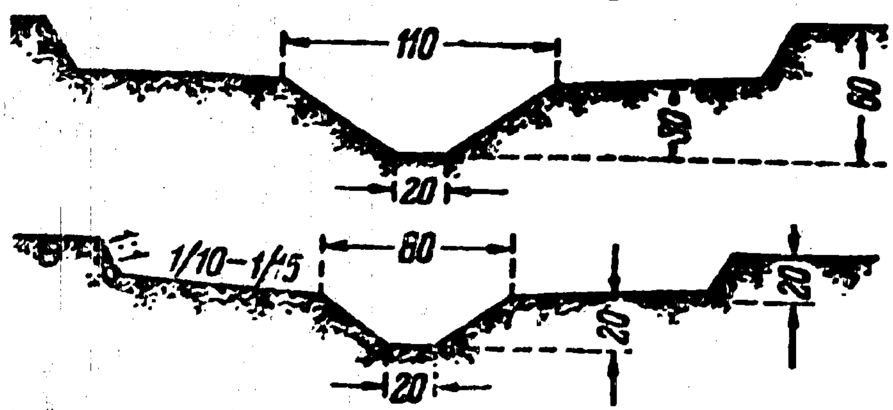


Fig. 124 - Optimum Profiles for Drainage Ditches

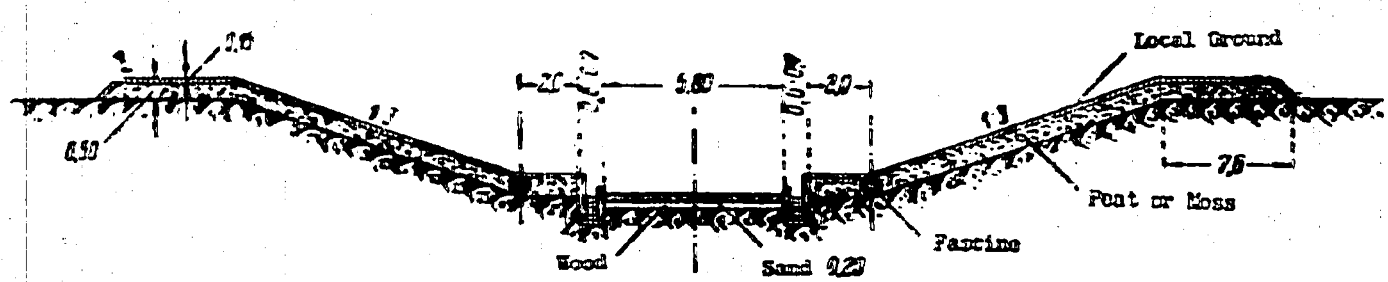


Fig. 125 - Cut in Excessively Wet Silty Permafrost (Design by A. Kurtinov)

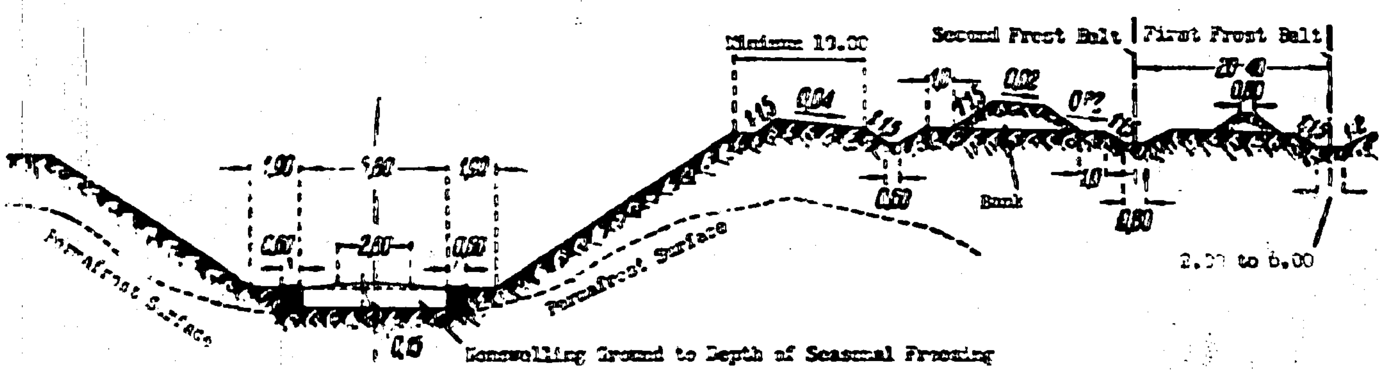
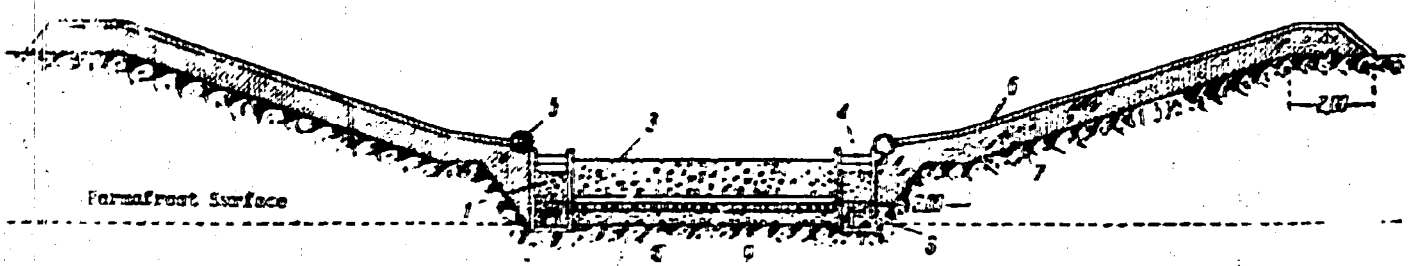


Fig. 126 - Cut in Swelling Ground (Design by A. Kurtinov)



1. Temporary insulation. 2. Pipe. 3. Well-draining material (rubble or coarse gravel). 4. Peat. 5. Fascine. 6. Mineral ground or rooted tundra sod. 7. Peat or moss. 8. Coarse gravel with intermediate layers of moss. 9. Peat layer 40 cm thick. 10. Double flooring of 10 by 10 cm beams.

Fig. 127 – Cut in Weak Silty Ground (Design by E. I. Sukhodolsky)

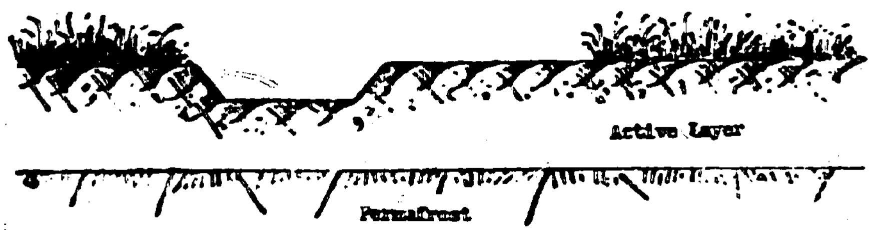


Fig. 128 – Frost Belt Consisting of Ditch and Wing

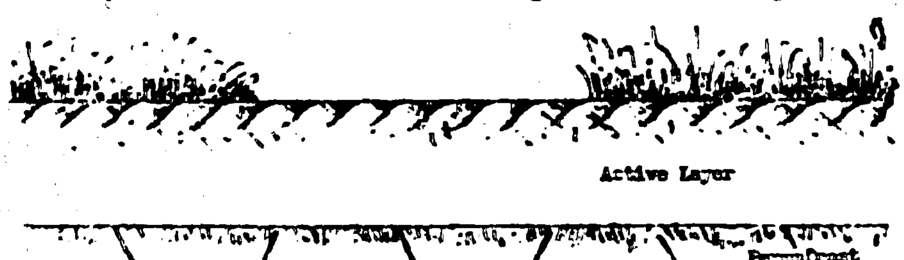


Fig. 129 – Frost Belt Consisting of Bared Ground Surface

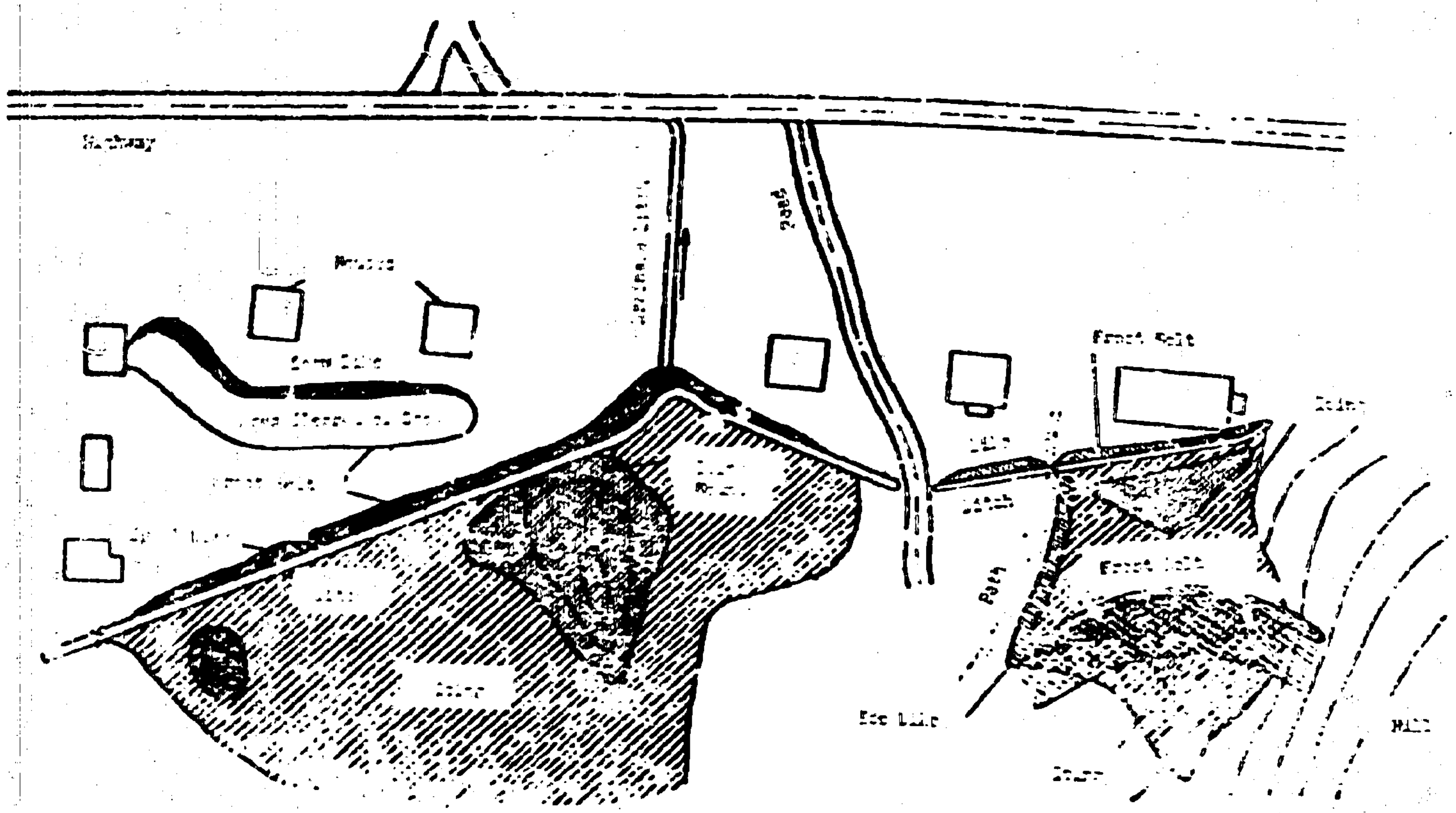


Fig. 138 - Plan of Icings and Frost Belts

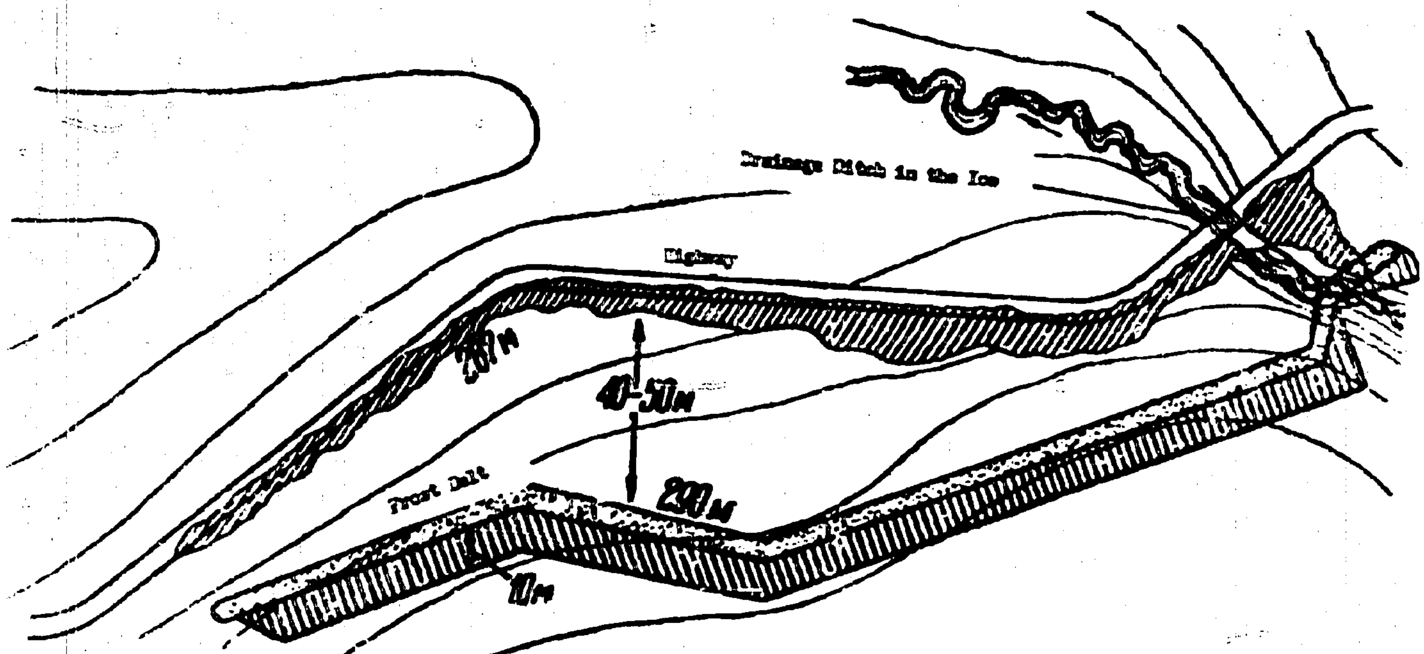


Fig. 131 - Frost Belt

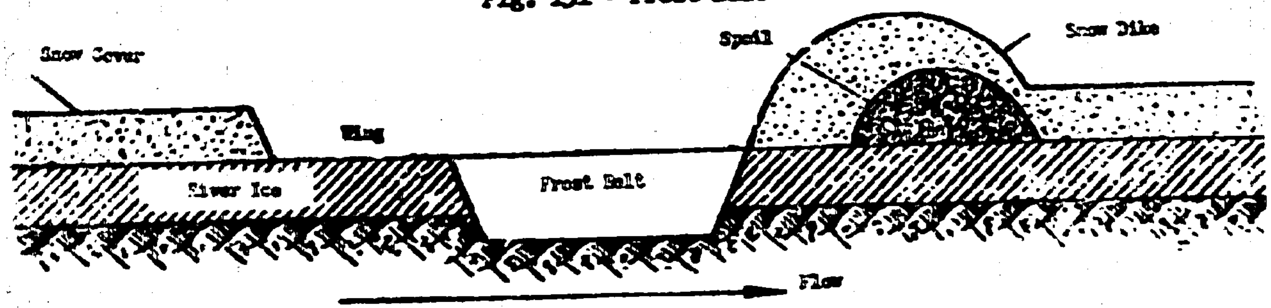


Fig. 132 - Frost Belt Across a River

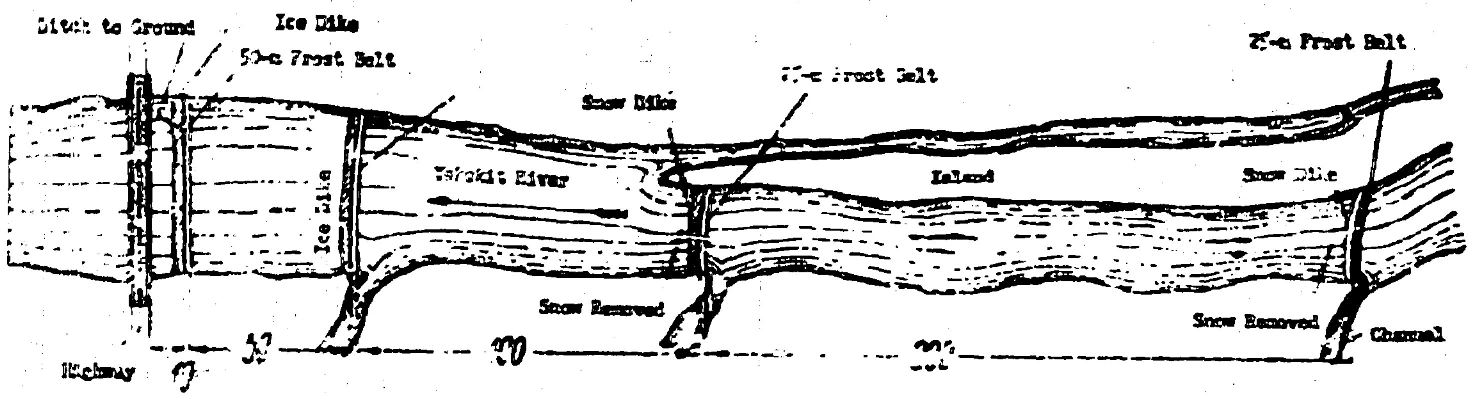


Fig. 133 - Frost Belts Across the Iakokit River

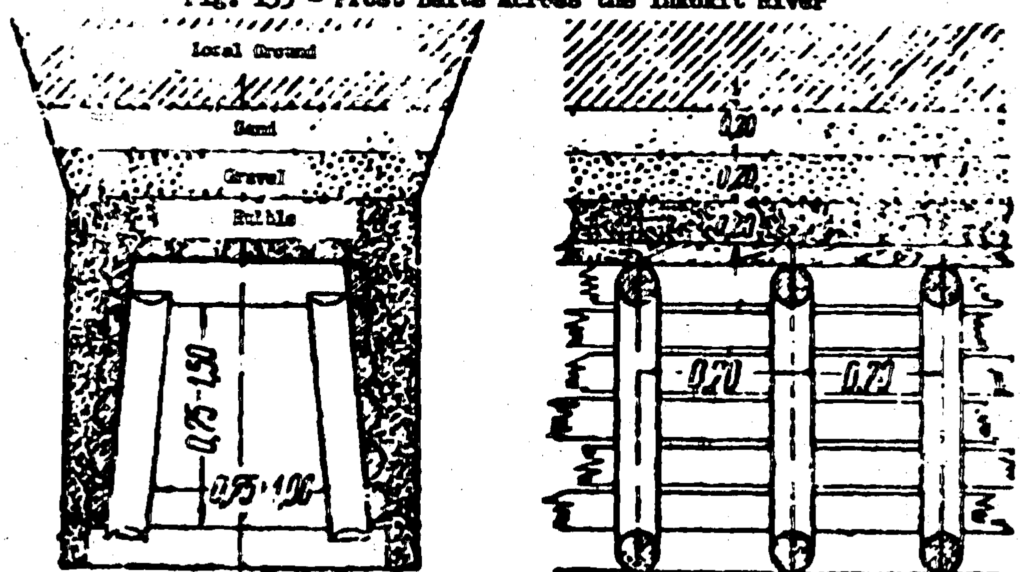


Fig. 134 - Underground Gallery for Capping Springs

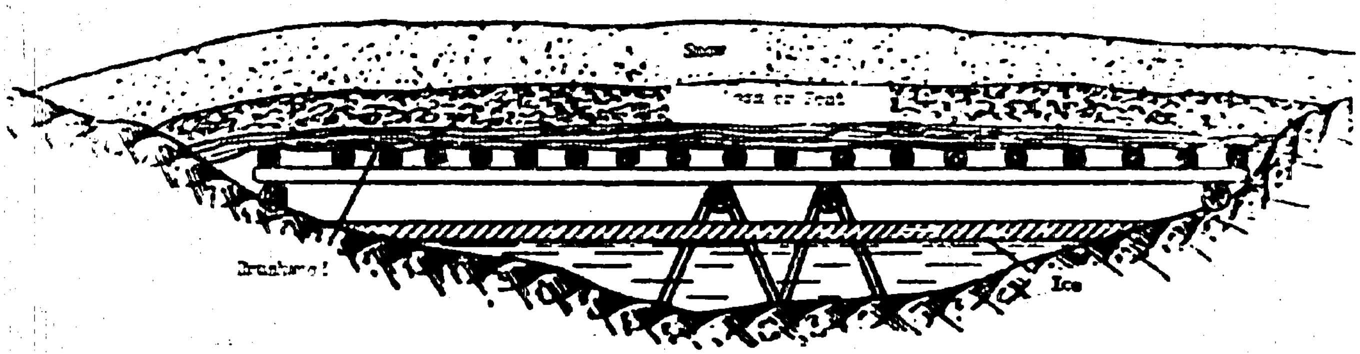


Fig. 135 - Insulation of a River Bed near a Bridge (Design by A. M. Chekotillo)

Fig. 136 - Temporary Road along the Amur Highway

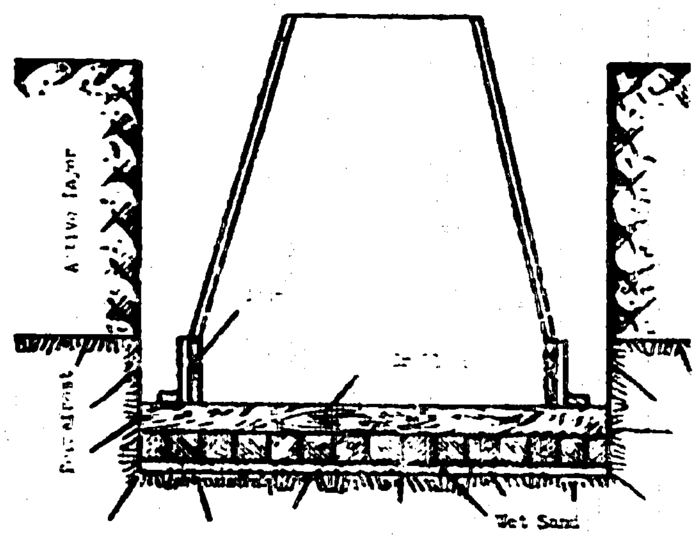


Fig. 137 - Typical Foundation Design

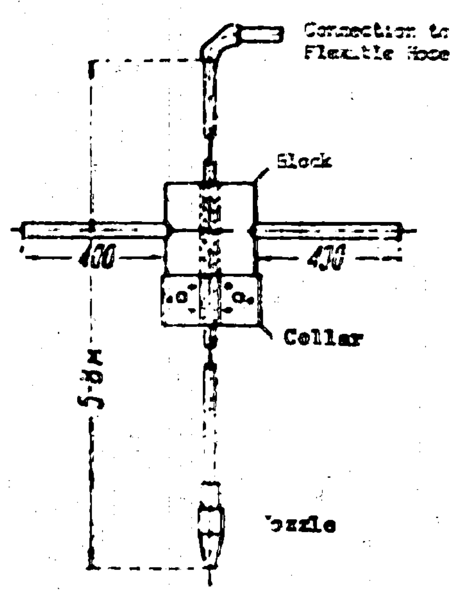


Fig. 138 - American Steam Point

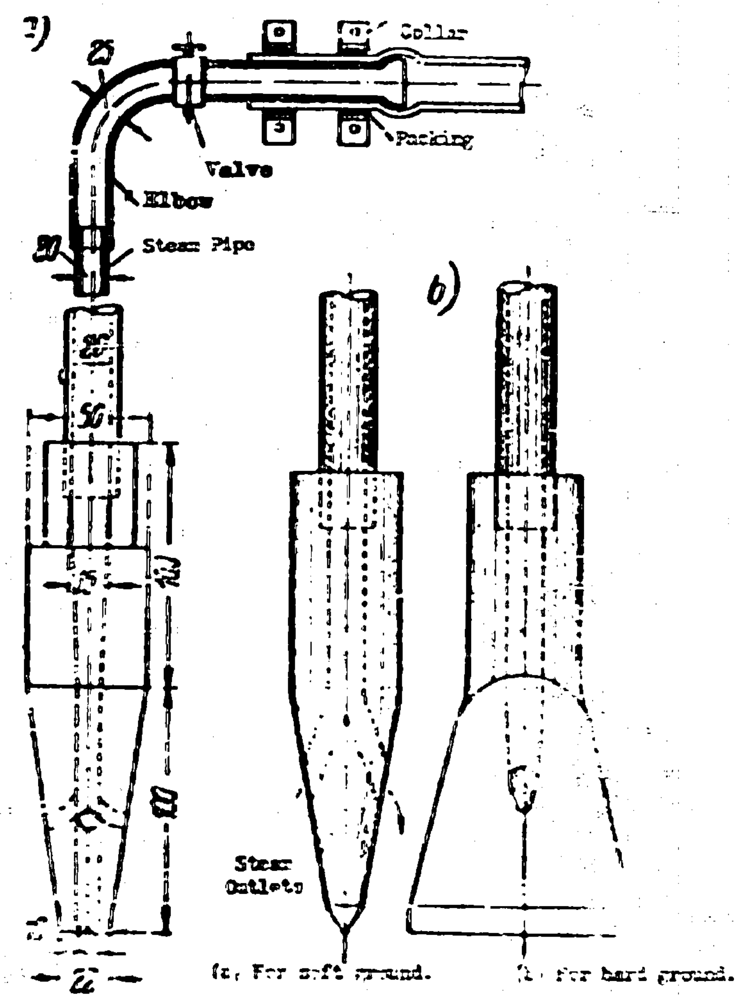


Fig. 139 - Types of Nozzles

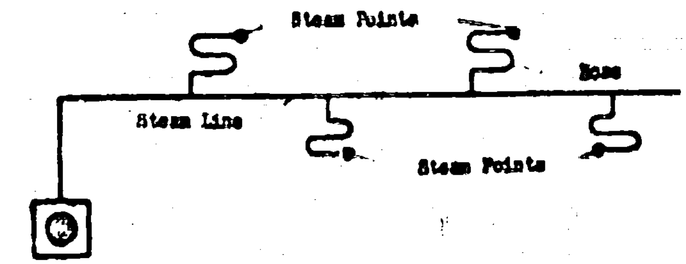


Fig. 140 - Steam Point Installation

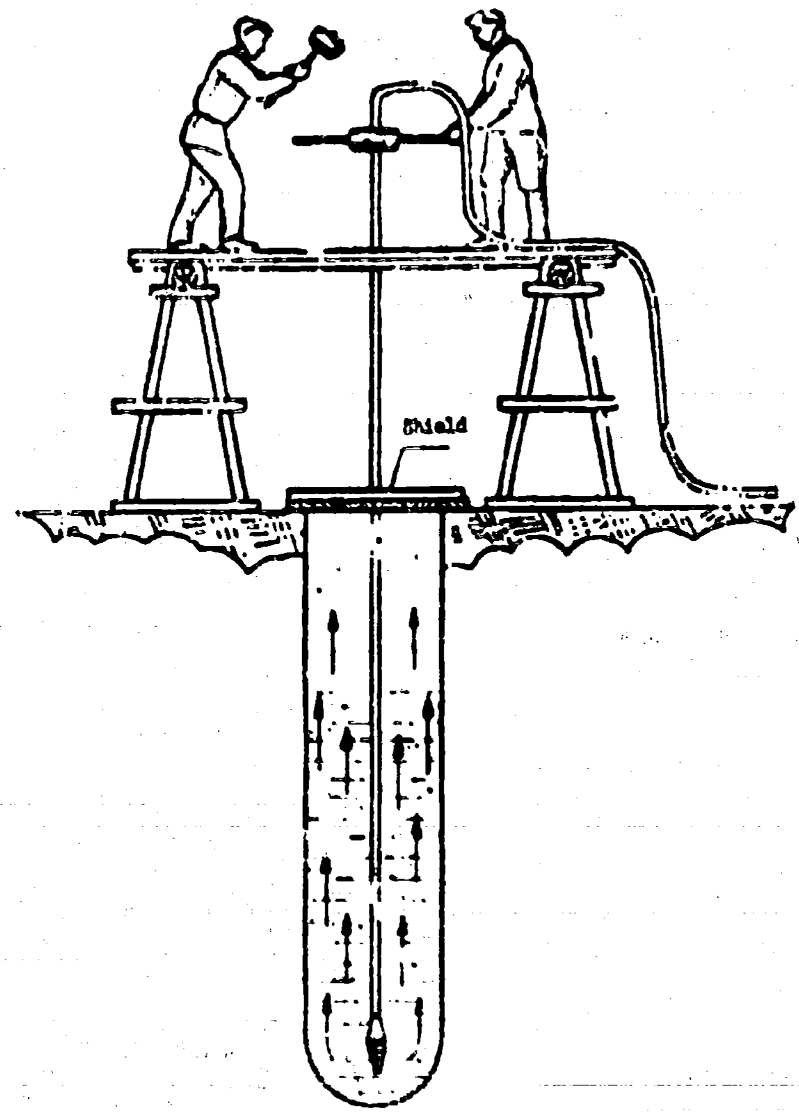


Fig. 141 - Steam Point Operation

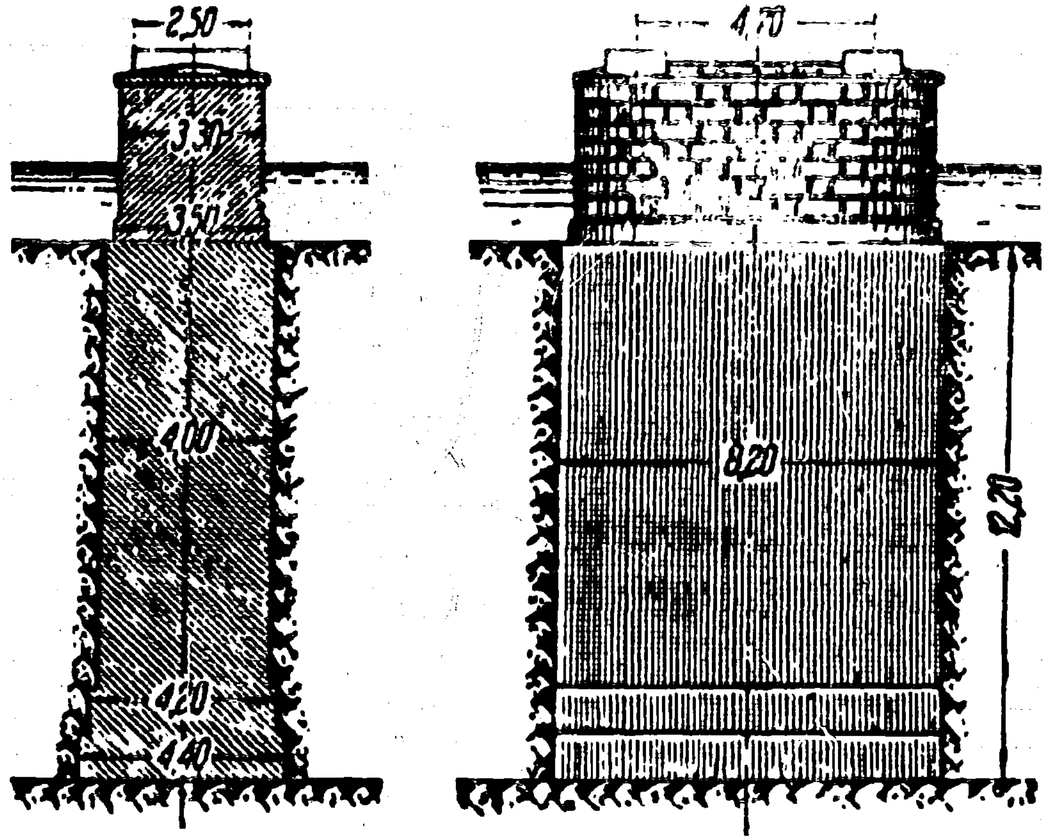


Fig. 142 - Center Pier of Bridge with 85-Meter Spans, Erected by Freezing Method

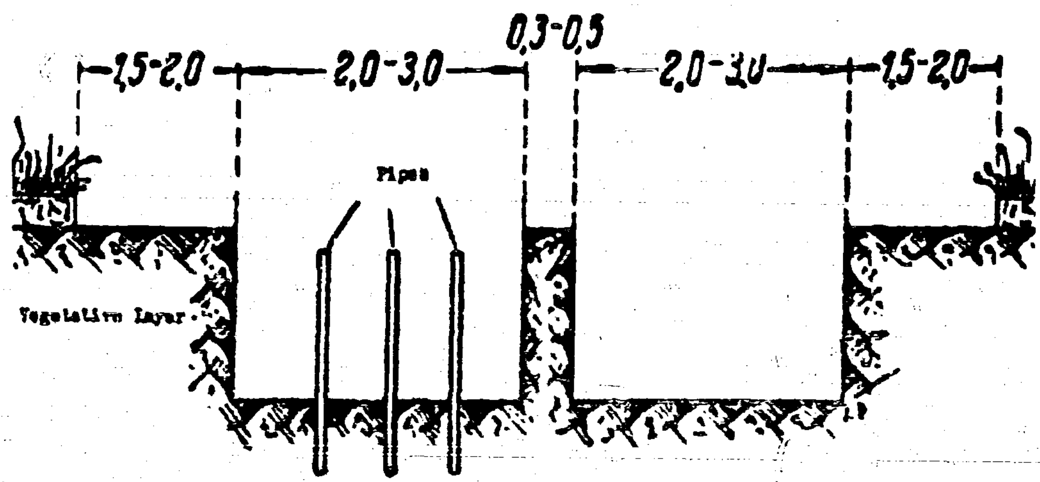


Fig. 143 - Diagram Illustrating Excavation by Freezing Method

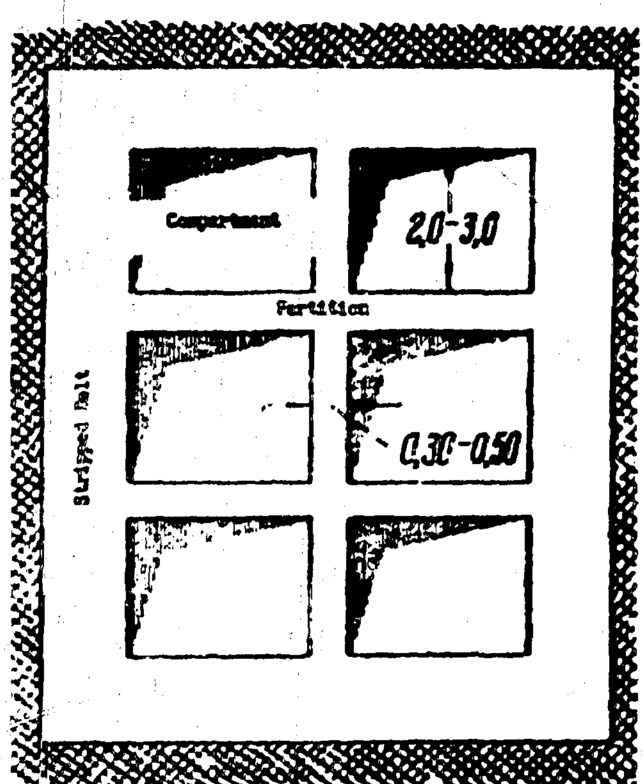


Fig. 144 - Plan of Pit Excavated by Freezing Method

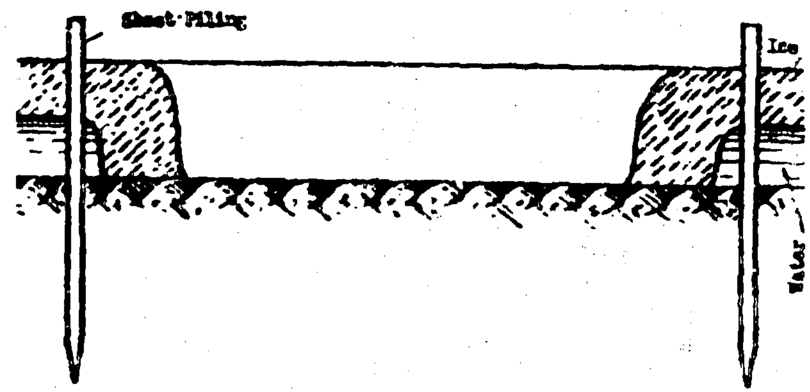


Fig. 145 - Freezing Method Applied to a Stream

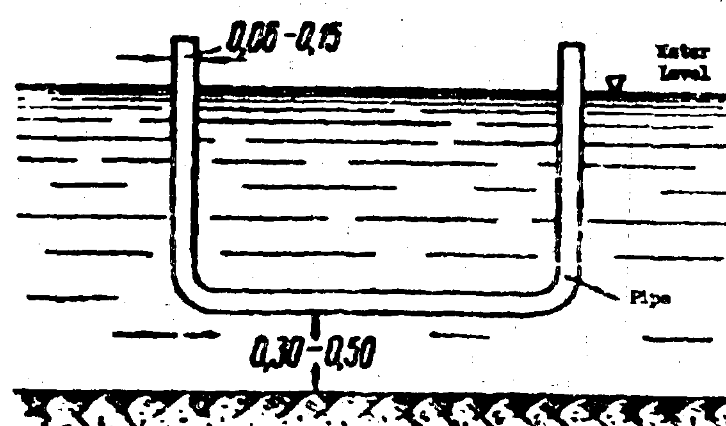


Fig. 146 - Freezing Method Utilizing Bent Pipes

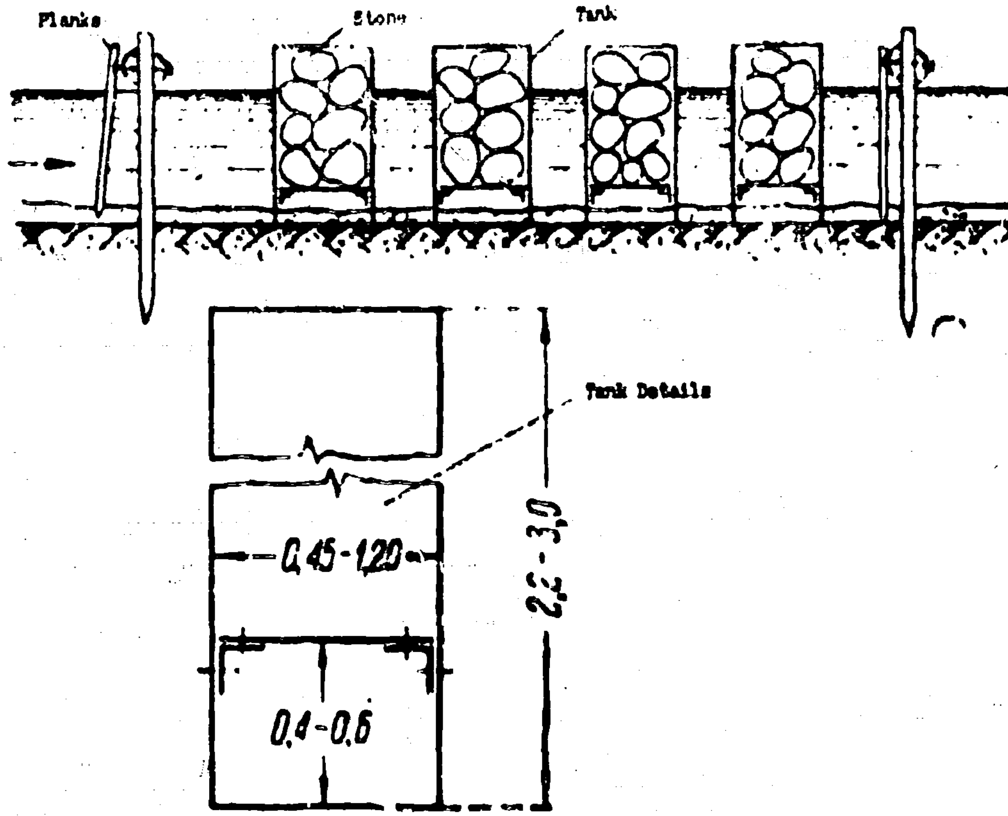


Fig. 147 - Excavation in a Stream by Freezing Method Utilizing Iron Tanks

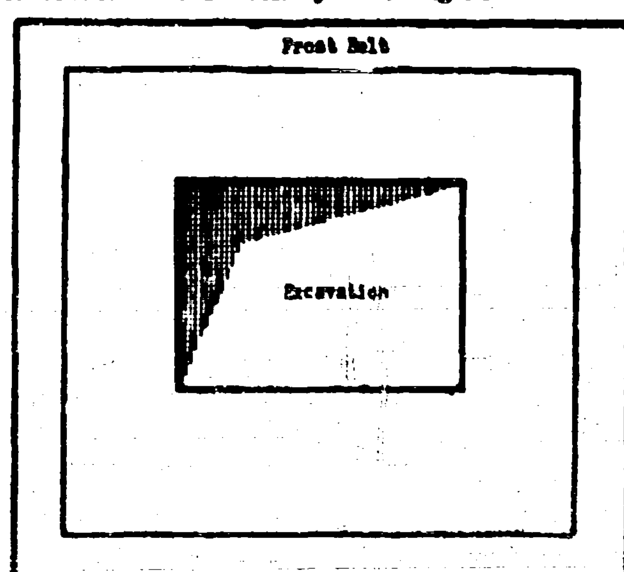


Fig. 148 - Use of Frost Belts To Prevent Water Seepage into an Excavation

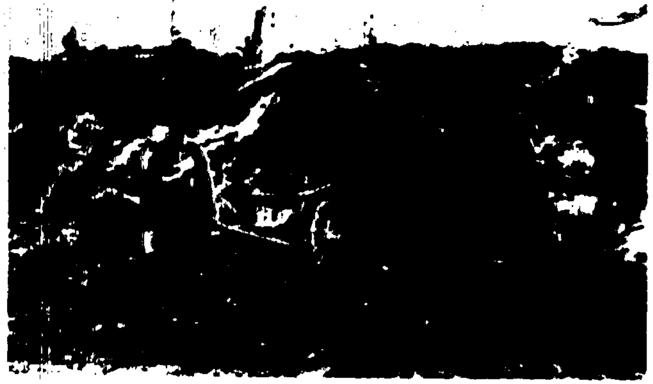


Fig. 149 - Arch of Frozen Ground

Fig. 151 - Water Accumulated in a Muddy Cut

Fig. 150 - Muddy Cut with Sliding Slopes

Fig. 152 - View of Excavation and Muddy Cut on the Amur Road

Fig. 153 - Thawing Operations with Bonfires

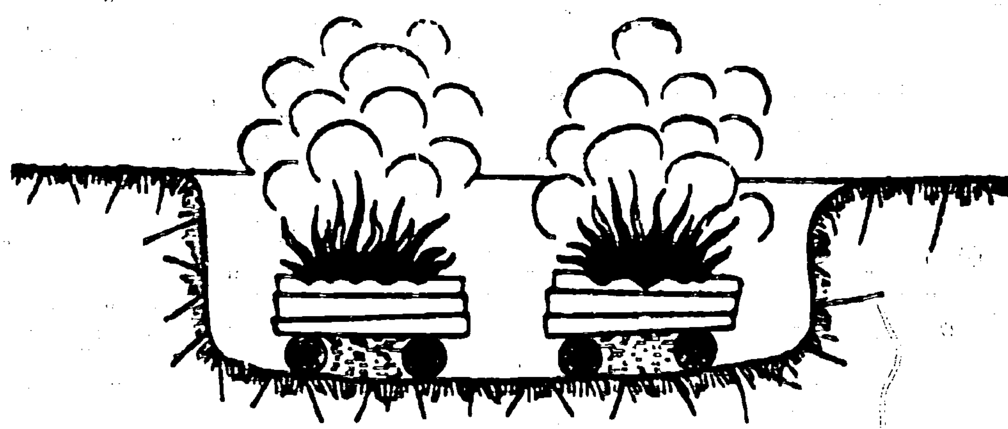


Fig. 154 - Open Bonfire in an Excavation

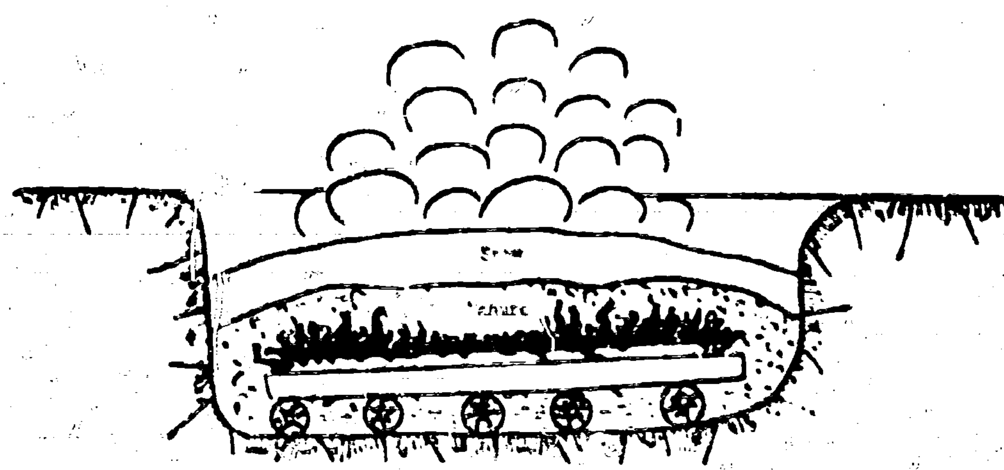


Fig. 155 - Covered Bonfire in an Excavation

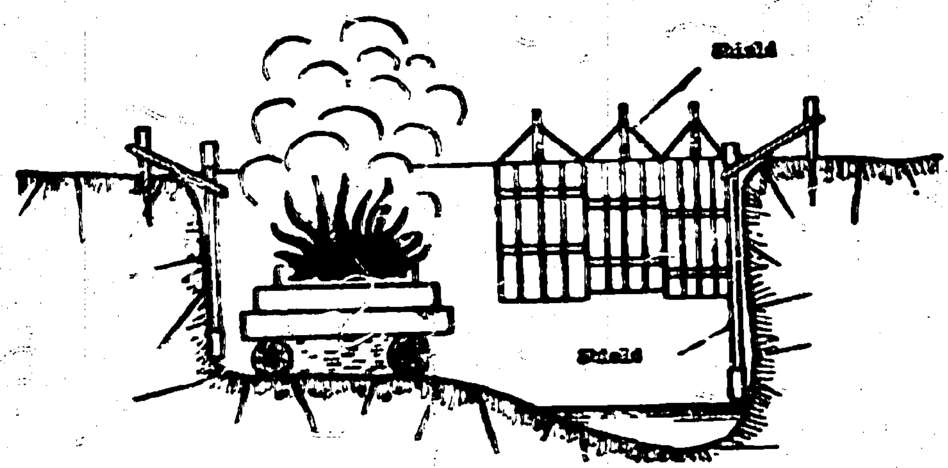


Fig. 156 - Wooden Shields Protecting Frozen Excavation Walls During Bonfire Operation

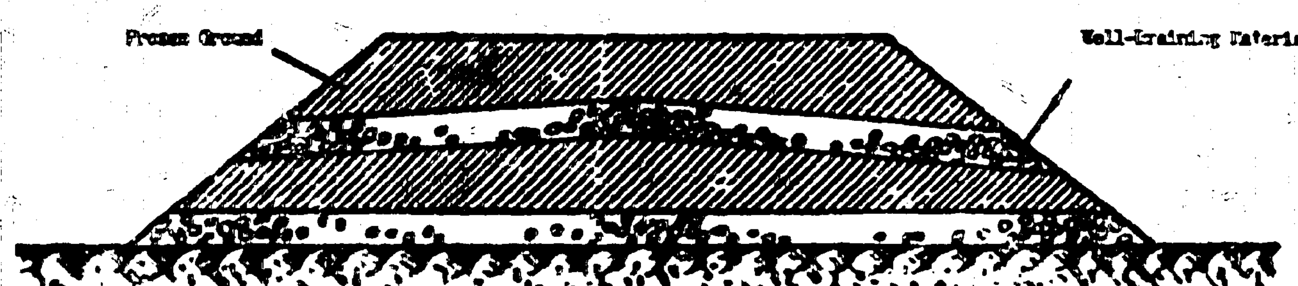


Fig. 157 - Design of Fill Comprising Frozen Ground